PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

(INSTITUTED 1852)

PAPERS AND DISCUSSIONS

VOL. XLIX

JANUARY TO DECEMBER, 1923

NEW YORK:
PUBLISHED BY THE SOCIETY

1923

Entered according to Act of Congress, in the year 1923, by the AMERICAN SOCIETY OF CIVIL ENGINEERS, in the office of the Librarian of Congress, at Washington.

NOTE.—This Society is not responsible for any statement made or opinion expressed in its publications.

Vol. XLIX. JANUARY, 1923. No. 1.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

CONTENTS

PAGE

Papers:

Analysis of Cost of Freight Service, Grand Trunk Railway Company of Canada. By J. P. NEWELL, M. AM. SOC. C. E. 3

The Design of Structural Supports for Turbo-Generators. By EDWARD H. CAMERON, ASSOC. M. AM. SOC. C. E. 37

Reports of Special Committees:

Final Report on Specifications for Design and Construction of Steel Railway Bridge Superstructure: Submitted by the Special Committee on Specifications for Bridge Design and Construction 53

Discussions:

The Water Power Problem. By MESSRS. J. B. LIPPINCOTT, C. S. JARVIS, D. C. HENNY, F. H. TIBBETTS, LOUIS C. HILL, C. H. BIRDSEYE, and EDWARD H. SARGENT 113

Engineering Geology of the Catskill Water Supply. By MESSRS. WILLIAM W. BRUSH, THOMAS H. WIGGIN, and JOEL D. JUSTIN. 123

Locomotive Loadings for Railway Bridges. By MESSRS. CHARLES EVAN FOWLER, C. F. LOWETH, RALPH MODJESKI, and C. A. P. TURNER 131

Tests of Concrete in Sea Water: Technical Papers Presented at the Annual Convention, Portsmouth, N. H., June 21st, 1922. By D. E. HUGHES, M. AM. SOC. C. E. 137

Memoirs:

ROBERT MOORE, PAST-PRESIDENT, AM. SOC. C. E. 139
IGNACIO MAREA DE VARONA, M. AM. SOC. C. E. 141
WILBUR FISK FOSTER, M. AM. SOC. C. E. 143
ROBERT CAMPBELL GEMMELL, M. AM. SOC. C. E. 150
MAURICE STILES PARKER, M. AM. SOC. C. E. 152
SAMUEL EVERETT TINKHAM, M. AM. SOC. C. E. 153
LUTHER WAGONER, M. AM. SOC. C. E. 155
FRANK SHERMAN WASHBURN, M. AM. SOC. C. E. 157
THOMAS DELANO WHISTLER, M. AM. SOC. C. E. 159
HERMAN DAVID HIRSCH, ASSOC. M. AM. SOC. C. E. 160
HUGO JULIUS SCHEUERMANN, ASSOC. M. AM. SOC. C. E. 160

For Index to all Papers, the discussion of which is current in *Proceedings*, see the second page of the cover

Vol. XLIX. JANUARY, 1923. No. 1.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

ANALYSIS OF COST OF FREIGHT SERVICE, GRAND TRUNK RAILWAY COMPANY OF CANADA

By J. P. Newell,* M. Am. Soc. C. E.

Synopsis

A part of the evidence presented to the Board of Arbitration appointed to determine the price to be paid by the Canadian Government for the preference and common stock of the Grand Trunk Railway Company was a study of the cost of freight service. For many years, the Grand Trunk Railway Company has sought traffic from Chicago, Ill., *via* Montreal, Ont., Canada, to New England ports, and even to New York City, in competition with the shorter American lines. This business is necessarily handled at high speed and low rates per ton-mile. The immediate purpose of the investigation was to determine whether this traffic is carried at a loss, but the methods used are here presented as applicable to the operations of any railroad, their specific application to the Grand Trunk Railway being used only for the purpose of illustration.

The general plan is one of analysis. The entire cost is assigned to various operations and, therefore, all unforeseeable happenings and accidents for the year are taken into account. The operations are divided into units, and the average cost of each is determined. In applying the results to specific cases, allowance is made for departures from average conditions. The year 1913 was selected as the latest one free from the influences of the World War.

Classification of Costs.—The cost of railroad transportation is composed of two principal elements: "Operating Expenses" and "Interest", including in the latter all charges of the same nature as interest. The Operating Expenses are divisible into Direct Costs, or those dependent on traffic, and Indirect Costs, or those not affected by traffic. Each class is also divided between the cost of freight and passenger service. The cost of passenger

Note.—Written discussion on this paper which will not be presented at any meeting of the Society, will be closed with the April, 1923, *Proceedings*. When finally closed the paper, with discussion, will be published in *Transactions*.

* Cons. Engr., Portland, Ore.

service has not been considered by the writer, except as was necessary in separating it from that of freight service.

The direct cost of freight service is divisible into two classes:

(1).—Terminal cost, or the expense of originating and delivering traffic independent of the distance moved. This cost has two subdivisions: (*a*) Carload; and (*b*) Less than Carload.

(2).—Road cost, or the expense of moving freight between the points of origination and delivery. This cost is divided between: (*a*) Distance, or the costs which are dependent on the distance moved; and (*b*) Resistance, or the costs which are dependent on the resistance overcome on account of friction and gradients.

Interest.—The interest charge is commonly regarded as "fixed", but, in reality, a considerable part is variable, increasing as traffic increases. This part is segregated and classified under the same heads as operating expenses.

Units of Service.—The next step is to determine the number of unit operations performed during the year, under each of the classes into which the costs have been divided.

The unit of Terminal Carload operations is a single terminal handling of a car, either of origination or delivery. The unit of Less than Carload operations is one terminal handling of a net ton.

The units of operation in the Distance factor are the train-mile, the car-mile, and the gross ton-mile.

The unit of Resistance is the resistance equivalent to that of lifting 1 ton 1 ft., or of 1 ft-ton.

In calculating the cost of any specific freight movement, the carload is taken as the unit. The service units connected with the movement are calculated, and the respective costs applied to obtain the total, which is then divided by the number of tons in the car, to ascertain the cost per ton, or ton-mile. The indirect cost must be borne by the existing traffic, whether it is large or small, and may be apportioned according to any desired unit of traffic.

Conclusions.—In 1913, 85% of the operating expenses and 30% of the interest was due to traffic. Passenger service was more profitable than freight.

Excluding less than carload freight, 28% of the direct cost of handling freight was terminal, 33% was proportional to the resistance overcome in moving trains, and 39% was due to the distance the freight was moved. The cost of handling an empty car was about 75% of that of a load. Increasing the number of tons per car is more effective in reducing cost than increasing the number of cars per train.

With proper study of the problem and with intelligent selection and comparison of data, the cost of transporting commodities can eventually be computed as accurately as the cost of producing them.

Division of Operating Expenses Into Direct and Indirect

The first step in the study of the cost of service is the segregation of those expenses which are not affected by the volume of traffic. They form a con-

stant charge against the existing business of the railroad, whether such business is large or small.

Some expenses are affected by variations in traffic expressed by one unit and not by others. For example, the cost of trainmen is affected by changes in train-miles, but not by car-miles. Traffic, as the word is used in this connection, is understood to be measured in the unit applicable to the account under consideration, and that the variations considered are in quantity only, without change in car or train loading, seasonal distribution, balance between opposite directions, or any other characteristic which would affect cost. This is necessary in order that the effect of variations in traffic may not be confused with the effect of changes in efficiency.

Only those variations in traffic which are of considerable extent, are taken into account. There are many minor variations in the volume of traffic, which have no perceptible effect on expenses. A few more passengers or tons of freight may be carried by the same equipment at an additional cost which is negligible, but with the growth of traffic, there comes a time when additional equipment and facilities must be provided, and more men and trains placed in service. There is then a sudden increase in cost of operation, which can not properly be attributed solely to the last additional traffic that made it necessary, but which must be spread over all the business handled. Before such a change takes place, the operating cost per unit of traffic is less than the normal cost, and afterward, for a time, it is more than the normal cost. Both extremes are disregarded in this paper. Operating expenses respond much less readily to a decrease in traffic than to an increase, and the relations determined herein are not intended to apply to such conditions.

Certain operating expenses are partly indirect, because they are due, in part, to natural causes, and, in part, to the effects of traffic. This is true of nearly all the maintenance-of-way expenses. The roadbed and all the structures are affected in varying degrees by weather, rust, and decay, as well as by wear due to the wheels passing over them. The effect of natural causes remains the same under all circumstances, but its ratio to the total expenses decreases with the growth of traffic.

Transportation expenses are partly indirect from another cause. At the beginning of operations, a certain initial equipment is required, whether or not any traffic exists. For a time, this equipment is only partly used, and a large part of the corresponding expenses is indirect. When the growth of traffic passes the point where the initial facilities are fully used, additional facilities must be provided, approximately in proportion to the increase of traffic, and, thereafter, the expenses of both the maintenance and operation of the equipment are entirely direct. Indirect transportation expenses, therefore, would ultimately disappear, if it was not for the fact that, with a growing system, the process is repeated with each addition of mileage so that although these expenses are entirely direct on one part of the system, they are largely indirect on other parts.

This consideration affects also the direct proportion of maintenance expenses, for it is obvious that if the existence of facilities has been made neces-

sary by the growth of traffic, their maintenance is also a function of traffic. It is for this reason that the maintenance of industrial tracks is considered to be a direct expense.

By ascribing the expenses of maintaining and operating partly used facilities and equipment to direct cost only in proportion to the extent to which they are used, there is charged to direct cost only the cost of maintaining and operating equipment and facilities used to approximately normal capacity. By this, it is not meant that each train is loaded with all that it can carry. In order to take care of fluctuations in the volume of business, there must be operated, at all times, equipment and facilities in excess of the average requirements. When volume of traffic reaches the point at which the excess of capacity over average requirements is only sufficient properly to take care of peak loads, the equipment is being used to its normal capacity, and further increase will demand added equipment.

In estimating the direct and indirect proportions of operating expenses, the writer has considered the various primary accounts separately or in groups of related accounts. Many of the accounts are obviously direct or indirect, either wholly or in large part. Others can be estimated only on the basis of judgment and experience. In dealing with some of these accounts, the writer has made use of a method recommended by a committee of the American Railway Engineering Association* for comparing maintenance-of-way expenses, but has extended it to include certain other accounts.

In general, the method consists in calculating the density of traffic and the expense per mile in the account under consideration, and observing the extent to which the expense on railroads of heavy traffic exceeds that on those of light traffic. For this purpose, thirty American railroads have been selected, the majority of which are in the northern part of the United States and operate under conditions fairly comparable to those of the Grand Trunk Railway. Roads, the business of which is confined principally to freight or to passenger service, have been excluded.

The volume of traffic and the expense for each railroad for a given year was found from the records of the Interstate Commerce Commission. In each case, the roads have been divided into two groups, those above the average density of traffic and those below it. Assuming that the difference in expense in the two groups is proportional to the difference in traffic density, the percentage of the given expense due to traffic at any given density, has been calculated. By this means, all comparisons between large and small volumes of traffic are confined to the same period of time, thus avoiding the complications due to rising and falling prices and wages.

I.—Maintenance of Way and Structures

In applying the method of analysis described, the chief difficulty is in choosing a unit of traffic that is related closely enough to the expense under consideration, and in equating the units in freight and passenger service. The Committee of the American Railway Engineering Association, previously mentioned, recommends that for consideration of track-maintenance expenses,

* *Proceedings*, Am. Ry. Eng. Assoc., 1913.

what it calls "Equivalent Ton-Miles", shall be used to measure the density of traffic. That Committee estimates the damage, per ton-mile, to track caused by passenger trains as being twice that of freight trains, and the damage from locomotives, as twice that of cars. Therefore, the passenger-locomotive ton-miles is multiplied by 4, the passenger-car ton-miles and the freight-locomotive ton-miles, by 2, and the products are added to the gross ton-miles of freight cars. The total is the equivalent ton-mileage. From consideration of a number of railroads, that Committee has derived a formula for determining the normal percentage of any expense which is direct for a given density of traffic, as follows:

Let X = amount of indirect maintenance per mile of track.
C = total maintenance per mile of track.
M = millions of equivalent ton-miles per mile of track.
I = increase in maintenance per mile for an increase of 1 in the value of M.
R = ratio of I to X.

then, $X + MRX = C$, and $\frac{100\,MR}{1+MR}$ = the percentage of C, which is direct. The equivalent ton-mileage of the Grand Trunk Lines in Canada for the year was found to be 21 585 000 000. On the basis of 3 118 miles of railroad, the value of M was 7, and on the basis of 3 825 miles of main tracks, it was 5.7.

The various primary accounts have been divided as follows:

6.—Roadway and Track.—A study of the statistics of the Interstate Commerce Commission for recent years, in which this account has been divided into two parts, shows that about 30% of it consists of the care of the right of way, which is not affected by adding more tracks. In compiling Tables 1 to 11, inclusive, the writer, therefore, has measured the density of traffic by dividing the equivalent ton-miles by the miles of main track, but for measuring the expenditure, 70% of the other main tracks has been added to the first, calling the sum "Equivalent First Main Track", and dividing this into the amount spent for maintenance. Side-tracks have not been included for the reason that investments in yard and industrial tracks are due to the increase in the traffic over that which existed when the road was built and, consequently, all maintenance expenses on those tracks must be considered as direct. The same thing is true, in part, of passing tracks. Table 1 shows the method of deducing a ratio from the records of the Interstate Commerce Commission. The equivalent ton-miles are calculated from the net ton-miles and the locomotive and car-miles.

The average density of traffic was 5.71 million equivalent ton-miles. The roads with a greater density had an average of 9.30 million, with an expense per mile of $778. Those with less density averaged 4.34 million and an expense per mile of $504. If a difference in traffic density of 4.96 million meant a difference in cost of $274, a difference of $M = 1$ would mean $55.

The part of the expense per mile of the second group, which was due to traffic, was $4.34 \times \$55 = \$239$. The remainder, \$265, was indirect, and is represented by X in the formula, whereas,

$$R = \frac{I}{X} = \frac{55}{265} = 0.21.$$

As $M = 5.7$, the direct percentage is,

$$\frac{100\,(5.7 \times 0.21)}{1 + (5.7 \times 0.21)} = 55\%.$$

The average for the three years, 1911, 1912, and 1913, was 60%, which was accepted as the true ratio. Fig. 1 illustrates Table 1.

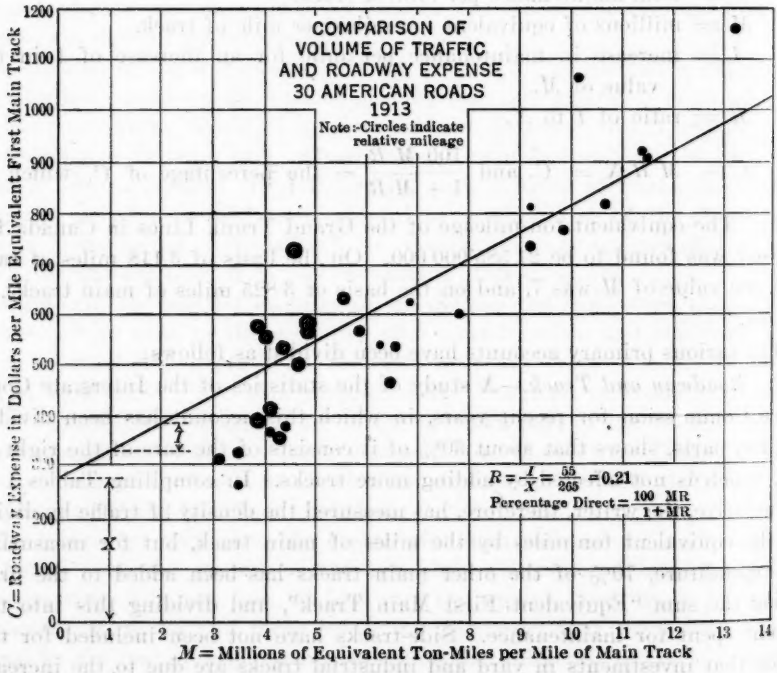


FIG. 1.

3.—Ties.—The cost and durability of ties varies so widely, and there are such differences in the practice of the various railroads in the preservation and protection of ties, that the writer has not relied on comparisons with other roads, but with the assistance of engineers familiar with the Grand Trunk Railway and the character of its maintenance, he has made estimates of the relative effects of traffic and natural causes on each section of the line. In this way, the tie maintenance of the System was classified as 34% direct.

4.—Rails and 5.—Other Track Materials.—In the case of these accounts, deterioration from any other cause than wear is so small that it may be ignored entirely, and the whole account treated as direct.

TABLE 1.—EFFECT OF VARIATIONS IN TRAFFIC ON ROADWAY AND TRACK MAINTENANCE, 1912.

Railroad.	Miles of railroad.	Miles of other main tracks.	Miles of equivalent first main track.	MILLIONS OF EQUIVALENT TON-MILES.		ROADWAY AND TRACK.	
				Total.	Per mile of main track.	Total.	Per mile of equivalent main track.
Pennsylvania	4 021	2 369	5 680	84 200	13.2	$6 603 000	$1 160
Philadelphia & Reading	1 015	620	1 450	18 800	11.5	1 322 000	910
Delaware, Lackawanna & Western	959	540	1 337	17 100	11.4	1 228 000	920
New York Central & Hudson River	3 595	2 338	5 282	63 200	10.7	4 303 000	820
Pittsburgh, Cincinnati, Chicago & St. Louis	1 467	783	2 015	22 700	10.1	2 164 000	1 070
Lake Shore & Michigan Central	1 775	1 196	2 612	29 300	9.9	2 010 000	770
Philadelphia, Baltimore & Washington	718	356	962	9 800	9.2	779 000	810
Erie	1 988	977	2 672	27 300	9.2	1 972 000	740
Cleveland, Cincinnati, Chicago & St. Louis	2 011	426	2 310	19 200	7.9	1 375 000	600
Chicago & Alton	1 025	193	1 160	8 400	6.9	726 000	630
Michigan Central	1 817	610	2 244	17 700	6.6	1 201 000	540
Wabash	2 515	100	2 585	17 100	6.5	1 209 000	470
Vandalia	827	71	877	5 700	6.3	471 000	540
Illinois Central	4 763	926	5 410	33 300	5.9	3 076 000	570
Sub-totals	28 491	11 505	36 546	373 800	9.30	28 439 000	778
Southern Pacific	6 310	333	6 543	37 300	5.6	4 194 000	640
Atchison, Topeka & Santa Fe	8 201	827	8 780	44 000	4.9	5 179 000	590
Louisville & Nashville	4 740	172	4 860	24 200	4.9	2 744 000	560
Union Pacific System	7 257	884	7 876	38 100	4.7	3 930 000	500
Central Pacific	10 988	754	11 510	54 400	4.6	8 392 000	730
Chicago, Burlington & Quincy	9 074	785	9 624	43 100	4.4	5 086 000	530
Grand Trunk Western	1 496	105	1 570	7 100	4.4	584 000	370
Chicago, Rock Island & Pacific	7 566	290	7 770	33 900	4.3	2 700 000	350
Chicago & North Western	7 960	1 068	8 708	37 300	4.1	3 529 000	410
Chicago St. Paul, Minneapolis & Omaha	1 744	129	1 834	7 600	4.1	659 000	360
Northern Pacific	6 420	624	6 857	28 500	4.0	3 831 000	560
Chicago, Milwaukee & St. Paul	7 511	680	7 987	32 200	3.9	3 111 000	390
Great Northern	7 482	201	7 623	29 700	3.9	4 401 000	580
Pere Marquette	2 330	276	2 523	9 100	3.5	682 000	270
Minneapolis, St. Paul & Sault Ste. Marie	3 773	59	3 814	13 000	3.4	1 226 000	320
Atlantic Coast Line	4 568	138	4 665	14 800	3.1	1 450 000	310
Sub-totals	97 415	7 325	102 544	454 300	4.34	51 698 000	504
Totals	125 906	18 880	139 090	828 100	5.71	.....	.....

9.—Bridges, Trestles, and Culverts.—The maintenance of culverts is not affected by traffic, and that of trestles very little. Steel bridges are very little affected by the elements, the principal indirect expense being for painting. The cost of retirements, which are due almost entirely to wear under traffic or to increase in weight of rolling stock, is direct. Records of the Grand Trunk Railway Company show retirements of steel bridges charged to operating expenses amounting to nearly one-half the total maintenance in 1913. The account, therefore, is estimated as 50% direct.

16.—Buildings, Fixtures, and Grounds.—Owing to the lack of any other basis for estimating the percentage of direct maintenance, recourse was had again to the records of other railroads. After much study, it was decided that equivalent ton-mileage although not directly affecting the maintenance of certain kinds of buildings, was, nevertheless, a sufficiently reliable standard by which to measure traffic density, and, accordingly, it was used in the same manner as it was for "Roadway and Track", except that miles of road and not of track were used for measuring both density and maintenance. For the three years, 1911-13, the records show an average value for R of 0.86. The value of M was 7.0, which makes the account 86% direct.

Other Expenses.—Of the remaining accounts under "Maintenance of Way", some are entirely indirect; some are small in amount, and have been assigned on somewhat arbitrary bases; and others are of a general nature and are apportioned according to the average of all accounts which can be directly assigned. They are combined in Table 2 as a single item.

II.—Maintenance of Equipment

The accounts under this group are nearly 100% direct. There is a slight reduction on steam locomotives, repairs, because of indirect train mileage (see Account 62, "Dispatching") and in freight cars, repairs, a small allowance is made for decay.

III.—Traffic

This group of accounts was estimated to be 52% direct.

IV.—Transportation Expenses

The expenses in this group are most readily apportioned by segregating them first, as between freight and passenger and then segregating each, as between direct and indirect. Practically the entire group would be direct, if it was not for branch lines on which the traffic is below the capacity of minimum service permissible. At the beginning of operation, each district must be given an amount of service adequate for the needs of the public, which will require only a small number of short trains. For a long time thereafter, the growth of traffic is taken care of by adding cars, with no change in the number of trains, and, consequently, no effect on those expenses dependent on train mileage.

Certain accounts are variable, chiefly according to car mileage. As soon, therefore, as it becomes necessary to add more cars, those expenses become wholly direct. Fuel, water, lubricants, and other supplies for road locomo-

tives, are partly, but not proportionately, increased by that growth of traffic which can be cared for by additional cars alone, and do not become wholly direct until increased train mileage becomes necessary.

62.—Dispatching.—This expense is apportioned between freight and passenger service on the basis of train mileage; in accordance with the Rules of the Interstate Commerce Commission, 56.3% is apportioned to freight.

It may be fairly assumed that as there was a large volume of travel on the main line, the passenger service was only that which was reasonably required to meet the demands of traffic, and that any substantial increase in traffic of the same character and having the same daily and seasonal distribution would have required additional train service. All main-line passenger-train expenses were, therefore, direct. The maximum number that could be averaged on branch-line passenger trains was taken as 4 cars, besides the baggage car; the actual average was 3.5 cars. These trains, therefore, were operated at 87.5% capacity, as it is fair to conclude that no more cars were hauled than the traffic required, although there is no doubt that the cars were only partly filled over much of the mileage. The two mileages were nearly equal so that of the two combined 93.5% was direct, and this percentage was applied to the dispatching of passenger trains.

Freight trains are also of two classes:

1.—Through freight trains on the main line and important branches: It is assumed that through freight trains are not sent out until sufficient tonnage is available to make up full trains. All these expenses are, therefore, direct.

2.—Way freights and mixed trains, constituting the whole service on many branches and a part of it on the main line: The latter is estimated to be 100% direct, as it is all greater than the minimum amount required to give sufficiently frequent service, and the trains no doubt are as heavy as practicable, considering the equipment used, the grades encountered, and the nature of the service. Branch-line way-freight trains were found to haul about three-fourths as many cars per train as main-line way freights. The latter trains both discharge and pick up freight while moving in either direction, but a branch-line train does nearly all its unloading while moving from the main line, and its picking up, in the opposite direction. This accounts for a part of the difference in train loading; therefore, the branch-line way freights have been estimated to be 80% direct. Mixed trains were estimated to be 38% direct.

The total freight-train mileage was 11 026 000, classified as follows:

Through freight	8 762 000	miles,	100%	Direct	=	8 762 000
Main line way	1 506 000	"	100%	"	=	1 506 000
Branch line way	446 000	"	80%	"	=	356 800
Mixed trains	312 000	"	38%	"	=	118 560
Total	11 026 000	miles,	97.4%	Direct	=	10 743 360

This makes the total mileage 97.4% direct, which percentage is to be applied to all freight-train expenses. Together with the percentage of 93.5 for passenger trains, it makes the account of "Dispatching," 95.5% direct.

63.—Station Employees.—At small stations, a minimum force must be employed, but the traffic is often insufficient to keep such a force at work to full capacity. Usually, some additional traffic could be cared for at any station, without increasing the number of employees proportionately. Only an intimate acquaintance with the station work on every part of the System would enable one to make a reliable estimate of the extent to which this is true. The writer, therefore, has had recourse again to what for convenience may be called the "A. R. E. A. formula". The proper units for measuring growth of traffic for this purpose are the ton and the passenger, which has been equated on the assumption that 2 passengers equal 1 ton.

In 1912, thirteen of the roads given in Table 1, with an average traffic of 23.08 million "equated" tons per mile, spent $1 370 per mile for station employees. The corresponding figures for seventeen other roads are 4.38 million "equated" tons and $455 per mile for station employees. This gives a value for R of 0.20. Similar calculations give values for R equal to 0.21 and 0.19, respectively, for 1911 and 1913. On the same basis, the value of M is 8.5, and the percentage of direct, therefore, is 63. If 1 passenger is assumed as equivalent to 1 ton, the result will be changed only 1 per cent.

At some of the stations, the employees are directly assignable to freight or passenger service. These are estimated at 80% direct, and the remainder fixed at 53%, in order to make the total 63%, as previously given.

66.—Station Supplies and Expenses.—By the same method, this account is fixed at 73% direct.

67 to 76.—Yard Expenses.—These expenses are dependent on the number of cars handled, and the entire group of accounts, therefore, is considered to be 100% direct.

80.—Road Enginemen.—Direct assignment from the payrolls shows this account to be 70.6% freight and 29.4% passenger. Each account is direct in the ratio determined for train mileage under "Dispatching". This makes the entire account 96.3% direct.

82.—Fuel for Road Locomotives.—Fuel is directly assigned to freight and passenger service in the accounts of the Company, in the ratio of 70% freight to 30% passenger. The consumption of fuel while the train is in motion is directly proportional to the tonnage moved. A considerable quantity of the fuel is consumed by stops, road-switching, and waiting at stations. Such losses, however, will increase even faster than the increase in traffic, as the number of stations will be increased and more delays will be caused by trains meeting. The only coal consumption which is not direct, is a part of that accompanying train service not used to capacity. The coal consumption of way freights per train-mile was about the same as that of through freights, the fuel used in stops, waits, and road-switching making up for the lighter tonnage. Fuel consumption in freight service was, therefore, direct in the same ratio as train mileage, or 97.4 per cent.

Branch-line passenger trains were assumed to use three-fourths as much fuel per mile as main-line passenger-trains, so that the fuel consumption was

calculated to be 94.3% direct. Application of the ratios thus found, to freight and passenger service gives 96.5% of the entire account as direct.

Water, Lubricants, and Other Supplies.—These accounts are given the same classification as "Fuel".

88.—Road Trainmen.—Assignment direct from the payrolls shows the ratio in this case to be 72.3% freight and 27.7% passenger. Each service is direct in the ratio found under "Dispatching", namely, passenger, 93.5% direct, and freight, 97.4% direct. Applying these ratios, the entire cost is 96.3% direct.

89.—Train Supplies and Expenses.—The expenses in this group consist chiefly of cleaning, lighting, lubricating, and icing cars. Each expense is variable practically in accordance with the number of cars handled, therefore, the whole account is direct.

99.—Loss and Damage, Freight.—This account is entirely direct.

V.—General Expenses

These expenses are usually thought of as practically a fixed charge, but this is far from true. As traffic increases, employees are added in all general offices, new departments are instituted, and larger salaries are paid. These expenses have been apportioned by the writer on the average of all other Accounts 1 to 105, inclusive, which gives 85% direct.

Table 2 summarizes the analysis thus far, showing that in 1913, 85% of the expenses of the Grand Trunk Railway Company was direct; that is, if the traffic had been 10% more or less, with the same character and distribution, the operating expenses would have been similarly changed 8.5 per cent.

The percentage of direct operating expenses is usually estimated at a much lower figure, but a study of the statistics of the Interstate Commerce Commission bears out these results. Indeed, if revenue is taken as the measure of tonnage, it will be found that the expenses of the larger carriers exceed those of the smaller ones in practically direct proportion to their traffic. The same conclusion must be drawn from the fact that operating ratios average approximately the same among roads of light and of heavy traffic. Revenue, however, does not accurately measure traffic, otherwise, one would be obliged to conclude that operating expenses are 100% direct.

Division of Operating Expenses Between Freight and Passenger Service

The rules of the Interstate Commerce Commission for separation of operating expenses cover this subject so completely that the writer will not discuss it, except where he has departed from those rules.

I.—Maintenance of Way and Structures

Direct.—The rules of the Interstate Commerce Commission divide track-maintenance accounts on the basis of fuel used in the two services. In the present case, this rule would give 70% for freight, but the writer thinks that equivalent ton-mileage, as used by the American Railway Engineering Association, furnishes a better basis, and it has been used for all railroad tracks,

thereby assigning only 60% to freight. Maintenance of yard tracks and sidings is divided according to locomotive switching mileage, 90% freight. The different classes of tracks are equated, in maintenance expenses, for each account, as follows:

Ties:

1 mile of main track = 2 miles of yard tracks = 1½ miles of passenger tracks.

Rails:

1 mile of main track = 4 miles of yard tracks = 2 miles of passenger tracks.

Other Track Material:

1 mile of main track = 1 mile of yard track = 1 mile of passenger track.

Roadway and Track:

1 mile of main track = 3 miles of yard tracks = 2 miles of passenger tracks = 1.4 miles of second tracks.

The calculations are omitted for the sake of brevity.

Indirect.—The indirect maintenance-of-way expenses are not only independent of the volume of traffic, but also of its nature. There is nothing about the decay of a tie or the removal of snow from the track which attaches the expenses to freight as against passenger traffic, or the reverse. These expenses, therefore, cannot be apportioned on any special basis. They must be assigned arbitrarily, and the best basis for such assignment would seem to be that of the results obtained in each service. These results are indicated by the revenues, provided rates are properly adjusted as between the two services. This adjustment is most nearly correct when the earnings in each branch of the service bear the same relation to the direct cost in that branch. The indirect maintenance-of-way expenses are apportioned therefore in the same ratios as the total direct operating expenses, namely, 72.5% to freight, and 27.5% to passenger.

Table 3 shows the separation between freight and passenger service for both direct and indirect. The same ratios have been used for both, except as noted, or in certain transportation accounts, where the assignment is affected by the fact that one service is more completely direct than the other.

Direct Freight Operating Expenses, Division Between Terminal and Road

Definitions.—Terminal expenses are those incurred in the origination and delivery of traffic, without regard to the distance it is moved. They consist of loading, unloading and accounting for freight shipments, switching cars to and from industrial sidings, making up trains, in short, of all expenses by which freight is placed on the main-line track, ready to move, or taken from its position in the train to the consignee. They are divided into two classes, Car Load and Less than Car Load. All expenses incident to the movement of car loads of less than carload freight are included in the first class. The

second contains only those expenses peculiar to less than carload. ("Terminal", as the word is used in this paper, does not refer to the ends of the road.)

TABLE 2.—ANALYSIS OF OPERATING EXPENSES, DIRECT AND INDIRECT, GRAND TRUNK RAILWAY OF CANADA, 1913.

Account.	Total.	Total.		
		Direct.		Indirect.
		Percentage.	Amount.	Amount.
I.—MAINTENANCE OF WAY AND STRUCTURES:				
3.—Ties	$1 014 237	34.0	$ 344 800	$ 669 437
4.—Rails	96 632	100.0	96 632	
5.—Other track material	203 855	100.0	203 855	
6.—Roadway and track	1 558 180	60.0	935 000	623 180
9.—Bridges, trestles and culverts	467 530	50.0	233 765	233 765
16.—Buildings	761 008	86.0	654 500	106 508
Other accounts	254 013	.....	36 307	217 706
Totals	$4 355 455	57.5	$ 2 504 859	$1 850 596
II.—MAINTENANCE OF EQUIPMENT:				
25.—Steam locomotives, repairs	$2 981 492	96.3	$ 2 871 000	$ 110 492
31.—Passenger cars, repairs	544 661	100.0	544 661	
34.—Freight cars, repairs	2 191 286	98.0	2 147 400	43 886
Other accounts	542 499	.....	468 800	73 699
Totals	$6 259 938	96.4	$ 6 031 861	$ 228 077
III.—TRAFFIC:				
All accounts	$1 217 251	52.0	$ 633 251	$ 584 000
IV.—TRANSPORTATION:				
62.—Dispatching	$134 257	95.5	$ 128 200	$ 6 057
63.—Station employees directly assigned	979 077	80.0	783 000	196 077
63.— " " apportioned	1 673 291	53.0	887 000	786 291
66.— " supplies and expenses	263 106	73.0	192 000	71 106
67-76.—Yard expenses	2 599 712	100.0	2 599 712	
80.—Road enginemen	1 718 703	96.3	1 654 300	64 403
82-85.—Fuel, etc., for railroad locomotives	4 651 143	96.5	4 488 400	162 743
88.—Road trainmen	1 991 130	96.3	1 917 000	74 130
89.—Train supplies and expenses	551 853	100.0	551 853	
99.—Loss and damage, freight	401 432	100.0	401 432	
Other expenses	1 069 434	.....	950 272	119 162
Totals	$16 033 138	91.0	$14 553 169	$1 479 969
Outside operations	$261 364	75.0	196 023	65 341
V.—GENERAL EXPENSES	1 062 028	85.0	903 000	159 028
Grand totals	$29 189 174	85.0	$24 822 163	$4 367 011

Road expenses are those incurred in the moving of freight between the stations of origin and destination. They are of two kinds, namely, those proportional to the distance the freight is moved, called Distance Expenses, and those dependent on the physical resistance to movement, or Resistance Expenses. Distance is subdivided, according to the unit directly affected, into train-mile, car-mile, and gross ton-mile.

TABLE 3.—(*Continued*)

Account.	Direct.				Indirect.			
	Total.	Freight.		Passenger.	Total.	Freight.		Passenger.
		Percentage.	Amount.	Amount.		Percentage.	Amount.	Amount.
IV.—Transportation:								
62.—Dispatching	$ 128 200	57.5	$ 73 700	$ 54 500	$ 6 057	34.0	$ 2 057	$ 4 000
63.—Station employees, assigned	783 000	89.5	701 000	82 000	196 077	89.5	175 154	20 923
63.—Station employees, apportioned	887 000	70.0	621 000	266 000	786 291	70.0	550 291	236 000
66.—Station supplies and expenses	192 000	53.0	102 000	90 000	71 106	53.0	37 700	33 406
67-76.—Yard expenses	2 599 712	90.0	2 340 000	259 712				
80.—Road enginemen	1 654 300	71.4	1 181 520	472 780	64 403	49.0	31 540	32 863
82.-85.—Fuel, etc., for road locomotives	4 488 400	70.8	3 177 700	1 310 700	162 743	48.0	78 230	84 513
88.—Road trainmen	1 917 000	73.0	1 400 000	517 000	74 130	53.0	39 400	34 730
89.—Train supplies and expenses	551 853	57.0	314 500	237 353				
99.—Loss and damage, freight	401 432	100.0	401 432					
Other expenses	950 272		606 985	343 287	119 162		59 985	59 177
Totals	$14 553 169	75.0	$10 919 837	$3 633 332	$1 479 969	66.0	$ 974 357	$ 505 612
Outside operations	$ 196 023	0	.....	$ 196 023	$ 65 341	0	.....	$ 65 341
V.—General Expenses	$ 903 000	72.5	$ 655 000	$ 248 000	$ 159 028	65.0	$ 103 000	$ 56 028
Grand totals	$24 822 163	72.5	$17 991 405	$6 830 758	$4 867 011	65.0	$2 842 665	$1 524 346

TABLE 3.—Analysis of Operating Expenses, Freight and Passenger.

Account.	Direct.				Indirect.			
	Total.	Freight.		Passenger.	Total.	Freight.		Passenger.
		Percentage.	Amount.	Amount.		Percentage.	Amount.	Amount.
I.—Maintenance of Way and Structures:								
3.—Ties	$ 344 800	70	$ 241 000	$ 103 800	$ 669 437	72.5	$ 485 000	$ 184 437
4.—Rails	96 632	62	60 000	36 632				
5.—Other track material	203 855	66	134 000	69 855				
6.—Roadway and track	935 000	64	600 000	335 000	623 180	72.5	452 000	171 180
9.—Bridges	233 765	60	140 000	93 765	233 765	72.5	169 500	64 265
16.—Buildings	654 500	70	458 000	196 500	106 508	72.5	77 208	29 300
Other accounts	36 307		18 700	17 607	217 706		157 980	59 726
Totals	$ 2 504 859	66	$1 651 700	$ 853 159	$1 850 596	72.5	$1 341 688	$ 508 908
II.—Maintenance of Equipment:								
25.—Steam locomotives, repairs	$ 2 871 000	71	$ 2 038 000	$ 833 000	$ 110 492	48	$ 53 000	$ 57 492
31.—Passenger cars, repairs	544 661	0		544 661				
34.—Freight cars, repairs	2 147 400	100	2 147 400		43 886	100	43 886	
Other accounts	468 800		329 450	139 350	73 699		44 295	29 404
Totals	$ 6 031 861	75	$ 4 514 850	$1 517 011	$ 228 077	62	$ 141 181	$ 86 896
III.—Traffic:								
All Accounts	$ 633 251	40	$ 250 018	$ 383 233	$ 584 000	48	$ 282 439	$ 301 561

Costs are not necessarily classified by their causes, but by the factors in accordance with which they are variable. For instance, the expense of enginemen is incurred solely on account of Resistance, but is assigned to Distance, as the men are paid according to the miles run.

I.—Maintenance of Way and Structures

2 to 6.—Track Accounts.—Yard tracks and sidings are used for both origination and delivery of freight, which is a terminal expense, and, at division points and junctions, for operations connected with through movements, which belong to distance expenses. No accurate apportionment can be made between the two classes, but it can be approximated closely enough for the purpose of this paper.

The amounts of freight originated and delivered at each division point were known and from calculations of tonnage moved east and west, the tons into and out of each point were ascertained. The respective percentages of local and through tonnage were then calculated for each division terminal. The trackage at each point was next measured. After deducting the local tracks, which include team tracks and those serving freight-houses, stock-yards, warehouses, and industries, the remainder was divided between through and local service according to the percentages stated previously. In this way, the tracks of Grand Trunk Railway were classified, as follows:

	Miles.
First main track	3 118
Second main track	707
Passing tracks	231
Yard tracks used in through service	217
Yard tracks used in local service	182
Local tracks and sidings	597
Total	5 052

These figures were used for subdividing track maintenance expenses.

3.—Ties.—Of the direct expense which was apportioned to freight, 42% was on account of yard tracks and sidings. The maintenance of that part of those tracks which is used in local service is terminal, whereas that of the remainder, being proportional to the weight of cars handled, is charged to gross ton-miles. On account of less traffic over them, the writer has estimated that local tracks and sidings require only two-thirds as much tie maintenance per mile as yard tracks. The proportions are, therefore, as follows:

	Terminal.	Gross ton-mile.
Yard tracks in through service	...	217
Yard tracks in local service	182	...
Local tracks and sidings ($597 \times \frac{2}{3}$)	399	...
Total	581	217

The 42%, therefore, is divided into terminal carload, 31%, and gross ton-mile, 11 per cent. The remaining 58% is apportioned according to the equivalent ton-mileage in freight service, which was:

Locomotive ton-mileage	3 747 000 000
Gross ton-mileage	9 268 000 000
Total	13 015 000 000

The former belongs to resistance, 17%, and the latter to gross ton-miles, 41 per cent. The entire account, therefore, is apportioned:

Terminal carload	31%
Gross ton-miles	52%
Resistance	17%

The other track accounts are treated in the same manner, but the results vary, because of different values assumed for local as compared to yard tracks, which are, as follows:

Rails	1	mile	of	local	tracks	= ⅓	mile	of	yard	tracks
Other track material	1	"	"	"	"	= ½	"	"	"	"
Roadway and track	1	"	"	"	"	= ⅓	"	"	"	"

Because of variations in the proportions of maintenance due to yard tracks and sidings under each account, there is further difference in the results. The percentages apportioned to each classification are shown in Table 5.

9.—Bridges, Trestles, and Culverts.—These accounts were apportioned on equivalent ton-mileage, as was done with road tracks: Gross ton-mile, 71%, and resistance, 29 per cent.

16.—Buildings, Fixtures, and Grounds.—Station and office buildings are apportioned in the same manner as "Station Employees".

Roadway buildings are apportioned in the same proportion as the track accounts and bridges, combined.

In the case of fuel stations, the ratio of direct freight fuel used in road service to all fuel charged to direct freight is 83 per cent. The same percentage of the maintenance of fuel stations is charged to resistance, and the remaining 17% is divided in the same manner as "Yard Expenses".

Water stations are apportioned in the same manner as "Fuel Stations".

Shops are apportioned in the same manner as maintenance of equipment.

Engine houses are apportioned to resistance according to the proportion of engine-house expenses-road to total direct freight engine-house expenses, 73 per cent, the remaining 27% to be divided the same as "Yard Expenses".

The apportionment of the various parts of the primary account gives, for the total: Terminal, less than carload, 27.8%; terminal, carload, 17.2%; train-miles, 3.2%; car-miles, 14.0%; gross ton-miles, 1.5%; and resistance, 36.3 per cent.

II.—Maintenance of Equipment

25.—Steam Locomotives, Repairs.—This expense is assigned according to locomotive mileage, whether road or yard. The former is 71% of the total, and this percentage, therefore, is charged to resistance. The remainder is

divided the same as "Yard Expense," according to the switching mileage in terminal and through service.

34.—Freight-Train Cars, Repairs.—Mr. B. F. Zercher, former Master Car Builder at the Angus Shops of the Canadian Pacific Railway Company, and recently holding a similar position on the Grand Trunk Western Railway, who has given special study to the causes of damage to freight cars, estimates that 65% of all damage occurs in yard handling, 3% is due to curvature and grades, and the remainder is about equally divided between the starting and stopping of trains and causes independent of track or handling. Accepting his estimate (except for a small increase in amount due to resistance of curvature and grades), the writer has divided the yard handlings between Terminal and Distance in proportion to switching-locomotive mileage (see "Yard Expenses"). The apportionment of the entire account is as follows:

Handling local cars in yards (Terminal)	38%
Handling through cars in yards (Distance)	27%
Starting and stopping trains (Distance)	15%
Miscellaneous causes (Distance)	15%
Effects of curvature and grades (Resistance)	5%

The distance expense belongs to car-miles.

III.—Traffic

All expenses under traffic (Accounts 53 to 60), are terminal, and one-fourth has been apportioned to less than carload, and three-fourths to carload.

IV.—Transportation

62.—Dispatching.—This is entirely a train-mile expense.

63.—Station Employees.—The greater part of these employees are engaged in terminal operations, the principal exceptions being telegraph operators and men engaged in feeding and watering stock, straightening and reloading shipments, and other work connected with through business and the accounting therefor. These expenses are estimated as 15% of the entire account, which is divided between train-miles and car-miles.

Of the 85% terminal, the greater part belongs to less than carload business. Carload business requires only accounting, billing, weighing, occasional loading and unloading, and a proportion of the general expenses of the station. The writer estimates this as about one-sixth of the terminal charge. The account is apportioned as follows: Terminal, less than carload, 70%; terminal, carload, 15%; train-miles, 8%; and car-miles, 7 per cent.

66.—Station Supplies and Expenses.—These expenses are apportioned in the same manner as "Station Employees".

67 to 76.—Yard Expenses.—On account of greater average distance to be moved, smaller number of cars to be handled at one time, and the necessary movements of the empty car, 1 ton delivered to consignee, to team, or transfer track, or to freight-house, has been estimated as requiring as much switching service as 2 tons passing from one division to another. Weighting the local tonnage at this ratio, the switching-locomotive mileage at each point has been

apportioned between local and through service. The former required 58.6%, and this percentage of the yard expenses, therefore, is charged to terminal carload. The remainder is assigned to car-miles.

80.—Road Enginemen.—Although enginemen are employed because of the resistance to be overcome, the expense on this account is variable in accordance with variations in the resistance only as far as it causes helper or assistant engines to be used. Freight-locomotive mileage exceeded freight-train mileage by 20.3%, therefore, this proportion is charged to resistance and the remainder to train-miles and terminal. The time spent in switching cars or waiting for less than carload freight to be loaded or unloaded is terminal. The percentage of time chargeable to each of these causes is estimated under Account 82 ("Fuel for Road Locomotives"), from which the following apportionment is made:

Train-miles (running)	42.0%
Train-miles (delays)	25.3%
Terminal carload (switching)	6.2%
Terminal, less than carload (waiting for less than carload)	6.2%
Resistance (helper service)	20.3%

82.—Fuel for Road Locomotives.—Fuel is consumed by road locomotives when engaged as follows: (1) Firing up; (2) pulling trains; (3) running down hill by gravity; (4) waiting at meeting points and terminal delays; and (5) waiting while freight is being loaded or unloaded.

Item (1) should be divided in the same proportions as the remaining uses and, therefore, may be omitted from this calculation; it amounts to about 3% of the total. Item (2) is clearly chargeable to resistance. Item (4) is indirectly dependent on resistance, as the greater the energy required, the more engine service is needed and the larger the waste of fuel in delays. It also varies with distance, as the longer the run the more time wasted in delays. The quantity of fuel thus consumed, however, is very small, and, therefore, the factor of distance is ignored herein. The fuel consumed while running down hill may be considered as an addition to that consumed while climbing the same grades, and charged to resistance. Fuel burned while less than carload freight is being loaded or unloaded, or cars picked up or set out, is a terminal expense.

In the following estimate of time consumed in various ways by freight-train locomotives, the time actually running is taken as 100, and is assumed to be the same for all trains. The velocity profiles of the main line from Chicago, Ill., to Portland, Me., show that, in each direction, 78% of the time is occupied in steaming and 22% in drifting. In making these calculations, it is estimated that an engine steams always at full capacity, shutting off steam entirely when the speed limit is reached, and drifting without steam until the speed is sufficiently reduced. The estimate of time consumed in other ways is based on observation and experience, but any probable error in it will not be large enough seriously to affect any conclusions reached in this paper.

The distribution of time of freight-train locomotives on the road is as follows:

	Through freight.	Way freight.
Working	78	78
Drifting	22	22
Delays at terminals and on account of other trains	65	40
Switching at stations	..	80
Waiting account handling less than carload	..	80
Total	165	300

The average running time between stations, as shown by velocity profiles, was 23.0 miles per hour. The direct freight-train mileage shown on page 11, therefore, required in actual running time:

Through freight	8 762 000 miles	381 000 hours
Way and mixed freight	1 981 360 "	86 000 "

The entire time of these trains, distributed according to the ratios given, was:

Steaming:	Through	381 000 × 0.78	297 200	
	Way	86 000 × 0.78	67 100	364 300
Drifting:	Through	381 000 × 0.22	83 800	
	Way	86 000 × 0.22	18 900	102 700
Delays:	Through	381 000 × 0.65	247 700	
	Way	86 000 × 0.40	34 400	282 100
Switching:	Way	86 000 × 0.80	.....	68 800
Handling less than carload:	Way	86 000 × 0.80	.....	68 800
	Total			886 700

In percentage, this amounts to:

Running:	Steaming	41%	
	Drifting	11.6%	52.6%
Standing:	Delays	31.8%	
	Switching	7.8%	
	Handling less than carload	7.8%	47.4%
	Total		100.0%

The approximate consumption of coal, of 12 000 B. T. U. per lb., by locomotives not working, has been given,* as follows; in pounds, per 1 000 sq. ft. of heating surface:

Firing up, per trip	510 lb.
Standing, per hour	121 lb.
Drifting, per hour	219 lb.

* *Bulletin 148*, Am. Eng. Assoc.

On the basis of 2 lb. of coal per hour per sq. ft. of heating surface, when steaming, the consumption while standing was 6% as much, and while drifting 11% as much. The fuel used while switching at stations is estimated at one-half as much per hour as when standing.

The percentages of fuel used in various ways are as given in Table 4.

TABLE 4.

	Percentage Rate		Percentage of fuel.
	Of time.	Of use of fuel.	
Steaming	41.0	100% = 41.000	84.5
Drifting	11.6	11% = 1.276	2.6
Delays at terminals and account of other trains	31.8	6% = 1.908	3.9
Switching at stations	7.8	50% = 3.900	8.0
Handling less than carload	7.8	6% = 0.468	1.0
Total	.....	48.552	100.0

This account, therefore, is apportioned: Terminal less than carload, 1%; terminal carload, 8%; and resistance 91 per cent.

Water, Lubricants, and Other Supplies.—These expenses are apportioned the same as "Fuel".

88.—Road Trainmen.—These expenses should be apportioned according to the time spent by engines in various operations, except that allowance should be made for part of a train crew being engaged in handling less than carload freight, while the remainder are switching:

Train-miles (running time plus waiting for other trains)	84.4%
Terminal carload (switching)	6.0%
Terminal less than carload	9.6%
Total	100.0%

89.—Train Supplies and Expenses.—All or nearly all these items are car expenses and are variable with the distance that freight is transported. The charge, therefore, belongs to car-miles.

99.—Loss and Damage, Freight.—To the extent that damages occur before freight is actually in transit, or after it has reached the station to which it is destined, they are a terminal expense. The writer estimates this proportion as 60%, of which two-thirds is less than carload. Of the 40% occurring in transit, 60%, is estimated as proportional to train-mileage. The entire account is: terminal less than carload, 40%; terminal carload, 20%; train-miles, 24%; and car-miles, 16 per cent.

V.—General Expenses

These expenses are apportioned according to the proportions of all other accounts. Table 5 shows the complete analysis of direct freight expenses.

TABLE 5.—(*Continued*).

Account.	Direct Freight.													
		Terminal.				Road.								
						Distance.								
	Total.	Percentage.	Less than carload.	Percentage.	Carload.	Percentage.	Train-mile.	Percentage.	Car-mile.	Percentage.	Gross ton-mile.	Percentage.	Resistance.
IV.—Transportation:														
62.—Dispatching	$73 700	....	.....	....	.....	....	$73 700	....	.....	....	.....	....	.....
63.—Station employees	1 322 000	70.0	$925 000	15.0	$198 000	8.0	106 000	7.0	$93 000	....	.....	....	.....
66.—Station supplies and expenses	102 000	70.0	71 400	15.0	15 300	8.0	8 160	7.0	7 140	....	.....	....	.....
67-76.—Yard expenses	2 340 000	....	.....	58.6	1 371 000	....	.....	41.4	969 000	....	.....	....	.....
80.—Road enginemen	1 181 520	6.2	73 260	6.2	73 260	67.3	795 000	....	.....	....	.....	20.3	$240 000
82-85.—Fuel, etc	3 177 700	1.0	32 130	8.0	254 010	....	.....	....	.....	....	.....	91.0	2 891 560
88.—Road trainmen	1 400 000	9.6	134 000	6.0	84 000	84.4	1 182 000	....	.....	....	.....	....	.....
89.—Train supplies and expenses	314 500	....	.....	....	.....	....	.....	100.0	314 500	....	.....	....	.....
99.—Loss and damage, freight	401 432	40.0	160 600	20.0	80 300	24.0	96 300	16.0	64 232	....	.....	....	.....
Other expenses	606 985	....	*8 100	....	7 164	....	134 060	....	91 361	....	.....	....	382 500
Totals	$10 919 837	12.6	$1 388 290	19.0	$2 083 034	22.0	$2 395 220	14.2	$1 589 233	....	.....	32.2	$3 514 060
V.—General Expenses	$655 000	9.0	$59 000	21.7	$142 000	14.0	$92 000	18.5	$121 000	4.5	$30 000	32.3	$211 000
Grand totals	$17 991 405	9.0	$1 631 090	21.7	$3 901 252	14.0	$2 503 820	18.5	$3 325 983	4.5	$809 200	32.2	$5 820 060

*Credit.

TABLE 5.—Analysis of Direct Freight Expenses.

Account.	Direct Freight.												
	Total.	Terminal.				Road.							
						Distance.							
		Percentage.	Less than carload.	Percentage.	Carload.	Percentage.	Train-mile.	Percentage.	Car-mile.	Percentage.	Gross ton-mile.	Percentage.	Resistance.
I.—Maintenance of Way and Structures:													
3.—Ties	$241 000	....	.....	31.0	$75 000	....	.....	....	.....	52.0	$125 000	17.0	$41 000
4.—Rails	60 000	....	.....	5.5	3 300	....	.....	....	.....	68.0	40 800	26.5	15 900
5.—Other track material	134 000	....	.....	19.0	25 500	....	.....	....	.....	60.0	80 400	21.0	28 100
6.—Roadway and track	600 000	....	.....	12.0	72 000	....	.....	....	.....	64.0	384 000	24.0	144 000
9.—Bridges	140 000	....	.....	....	.....	....	.....	....	.....	71.0	99 400	29.0	40 600
16.—Buildings	458 000	27.8	$127 300	17.2	78 700	3.2	$14 700	14.0	$64 200	1.5	6 900	36.3	166 200
Other accounts	18 700	....	*6 000	....	*4 000	....	1 900	....	4 900	....	20 600	....	1 300
Totals	$1 651 700	7.3	$121 300	15.2	$250 500	1.0	$16 600	4.0	$69 100	46.0	$757 100	26.5	$487 100
II.—Maintenance of Equipment:													
25.—Steam locomotive repairs	$2 038 000	....	.....	17.0	$346 000	....	.....	12.0	$245 000	....	.....	71.0	$1 447 000
34.—Freight cars, repairs	2 147 400	....	.....	38.0	816 000	....	.....	57.0	1 224 000	....	.....	5.0	107 400
Other accounts	329 450	....	.....	....	76 200	....	.....	....	127 650	....	$22 100	....	108 500
Totals	$4 514 850	....	.....	27.4	$1 238 200	....	.....	35.4	$1 596 650	0.5	$22 100	36.7	$1 657 900
III.—Traffic:													
All accounts	$250 618	25.0	$62 500	75.0	$187 518	....	.....	....	.....	....	.....	....	.....

*Credit.

Interest on Investment

Certain parts of the investment in railroad properties increase about in proportion with the increase of traffic. The principal items thus affected are equipment, buildings, yard and individual tracks, and second tracks. Interest on these investments is, therefore, a part of the direct cost, and has been distributed in the same manner as the operating expenses. The average cost of money to the Grand Trunk Railway Company has been estimated at 5%, including discount and commissions.

Equipment.—The investment in equipment was estimated, as follows:

Passenger locomotives	$ 3 736 000
Freight "	10 651 000
Passenger cars	7 357 000
Freight cars	22 575 000

Locomotives.—The interest on investment in passenger locomotives is $186 800, of which, on the basis of passenger train-mileage, 93.5%, or $174 700, was direct. On freight locomotives, the interest was $532 550, of which 97.4%, or $518 700, was direct. This is divided on the basis of road and switching mileage, the same as locomotive repairs.

Passenger Cars.—Interest on this investment is $367 850, all direct.

Freight Cars.—The interest on this investment is $1 128 750, all direct, and is divided on the basis of time consumed in loading and unloading as compared with that in transit, which has been estimated as 60% and 40%, respectively. The amount, therefore, is apportioned as follows, terminal carload 60%; and car-mile, 40 per cent.

Buildings.—It was estimated that the investment in buildings was $17 000 000, and that the buildings were used to 90% of their capacity. The interest was, therefore, $850 000, of which $765 000 was direct. This is apportioned the same as the maintenance of buildings, fixtures, and grounds.

Yard Tracks and Industrial Sidings.—The investment in these tracks is estimated as 996 miles at $8 000 per mile, or roughly as $8 000 000, the interest on which would be $400 000. On the basis of locomotive switching, 90% belongs to freight. It has been shown that 217 miles of 996, or 22%, were used in through service. This percentage of the interest, therefore, is charged to distance, as a car-mile expense, and the remainder to terminal carload.

Second Tracks.—The Grand Trunk Railway Company has 707 miles of second track, but it appears that the greater part of this investment was not made necessary by the growth of traffic, and the inclusion of interest thereon as a direct charge on operations, therefore, would cause the relation of cost of service to growth of traffic to be incorrectly represented.

Hire of Equipment.—This item may properly be included with interest, as a charge of the same nature. The amount, $554 000, in the absence of records by which it could be directly assigned, has been divided between freight and passenger in the same ratio as the investment in cars, namely, 75% freight, 25% passenger, and the former has been apportioned in the same manner as interest on freight cars.

Indirect Interest.—In this study, as presented to the Board of Arbitration, only interest actually paid was considered, no account being taken of investment represented by stock. In order that the true relation between direct and indirect costs may be shown, the entire investment must be considered, as interest, or return on it, is as much a part of the cost of service as are the operating expenses.

The books of the Grand Trunk Railway Company show on December 31st, 1913, an investment in road and equipment of $380 000 000, or $122 000 per mile. Much of the property of the Grand Trunk was acquired in exchange for stock, so that the figures given do not represent the true investment. An approximate estimate is sufficient for the purpose of this paper, and, therefore, it has been taken at $250 000 000, or $80 000 per mile. The investment already assigned amounts to $70 000 000. The interest on the remainder is $9 000 000, which is divided between freight and passenger service in accordance with the sum of all direct costs, namely, freight, 73%, and passenger, 27 per cent. The results of the analysis of interest appear in Tables 6 and 7.

Taxes.—Taxes, amounting to about $475 000 per year, are an indirect cost and are apportioned on the same basis as indirect interest, namely, $347 000 to freight and $128 000 to passenger service.

Table 8 summarizes the cost of service. A comparison of earnings with cost in each service shows the passenger service to have been the more profitable. In 1913, freight revenues were $27 930 000, and passenger revenues, $14 280 000. The direct cost of freight service was 75% of the revenue, and that of passenger service was 55 per cent. Compared with the entire cost, the percentages were 110% for freight and 83% for passenger service.

Train-mile costs are shown to have been only about 60% as much as car-mile costs. A reduction of car-miles by heavier loading, therefore, will do more to effect economy than hauling more cars per train. The largest field for saving is in terminal operations which cost more than train-miles and car-miles combined. The direct cost was twice as much as the indirect cost.

Unit Costs

The direct cost of each unit of operation is next to be determined.

Terminal Less Than Carload.—The records of the Grand Trunk Railway Company afford no basis for estimating less than carload tonnage, but a similar analysis of cost of service of the Grand Trunk Western Railway Company gave 85 cents per ton for less than carload and one of the Atlantic and St. Lawrence Railway Company gave 80 cents per ton.

Terminal Carload.—The series of movements by which a car of freight is handled terminally has been classified, as follows:

At Point of Origin:

1.—Empty car from yard to shipper.
2.—Loaded car from shipper to yard.
3.—Loaded car from yard to train.

At Destination:

4.—Loaded car, train to yard.
5.—Loaded car, yard to consignee.

TABLE 6.—INTEREST ON INVESTMENT.

	Total.	Total:		Direct:			
		Direct.	Indirect.	Percentage.	Freight.	Percentage.	Passenger.
Passenger locomotives	$186 800	$174 700	$12 100	....	.....	....	$174 700
Freight locomotives	532 550	518 700	13 850	....	$518 700	....	.....
Passenger cars	367 850	367 850	.....	....	.....	....	367 850
Freight cars	1 128 750	1 128 750	.....	....	1 128 750	....	.....
Buildings	850 000	765 000	85 000	70	595 500	30	229 500
Yard tracks and industrial sidings	400 000	400 000	.....	90	360 000	10	40 000
Hire of equipment	554 000	554 000	.....	75	415 500	25	138 500
Other interest	9 000 000	.....	9 000 000	....	.....	....	.....
Totals	$18 019 950	$3 909 000	$9 110 950	....	$2 958 450	....	$950 550

TABLE 7.—INTEREST ON INVESTMENT, DIRECT FREIGHT.

	Total.	Terminal:				Road:							
		Less than carload.		Carload.		Train-mile.		Car-mile.		Gross ton-mile.		Resistance.	
		Percentage.	Amount.	Percentage.	Amount.	Percentage.	Amount.	Percentage.	Amount.	Percentage.	Amount.	Percentage.	Amount.
Freight locomotives	$518 700	....	.....	17.0	$88 300	....	.....	12.	$62 400	....	.....	71.0	$368 000
Freight cars	1 128 750	....	.....	60.0	677 250	....	.....	40.	451 500	....	.....	....	.....
Buildings	585 500	27.8	$149 000	17.2	92 000	3.2	$17 100	14.	75 000	1.5	$8 000	36.3	194 400
Yard tracks and industrial sidings	360 000	....	.....	78.0	281 000	....	.....	22.0	79 000	....	.....	....	.....
Hire of equipment	415 500	....	.....	60.0	249 300	....	.....	40.0	166 200	....	.....	....	.....
Totals	$2 958 450	5.0	$149 000	46.9	$1 387 850	0.6	$17 100	28.2	$834 100	0.8	$8 000	19.0	$562 400

TABLE 8.—Cost of Service

Direct	$28 731 163	67.3%	
Indirect	13 952 961	32.7%	
Total	$42 684 124		
Freight	$30 790 520	72.1%	
Passenger	11 893 600	27.9%	
	$42 684 124		

	Direct.				Indirect.			
	Freight:		Passenger:		Freight:		Passenger:	
	Percentage.	Amount.	Percentage.	Amount.	Percentage.	Amount.	Percentage.	Amount.
Operating expenses	72.5	$17 991 405	27.5	$6 830 758	65.0	$2 842 665	35.0	$1 524 346
Interest	75.7	2 958 450	24.3	950 550	73.0	6 651 000	27.0	2 459 950
Taxes	.....	.....	.....	.....	73.0	347 000	27.0	128 000
Totals	73.0	$20 949 855	27.0	$7 781 308	70.5	$9 840 665	29.5	$4 112 296

Direct Freight

	Terminal.				Road.							
	Less than carload.		Carload.		Train-mile.		Car-mile.		Gross ton-mile.		Resistance.	
	Percentage.	Amount.	Percentage.	Amount.	Percentage.	Amount.	Percentage.	Amount.	Percentage.	Amount.	Percentage.	Amount.
Operating expenses	9.0	$1 631 090	21.7	$3 901 252	14.0	$2 508 820	18.5	$3 325 983	4.5	$809 200	32.3	$5 820 060
Interest	5.0	149 000	46.9	1 387 850	0.6	17 100	28.2	834 100	0.3	8 000	19.0	562 400
Totals	8.5	$1 780 090	25.3	$5 289 102	12.0	$2 520 920	19.8	$4 160 083	3.9	$817 200	30.5	$6 382 460

TABLE 6.—Interest on Investment.

6.—Empty car, consignee to yard.

7.—Empty car, yard to train (for return).

At Point of Origin:

8.—Empty car, train to yard.

All these operations are considered as equal in cost, and the sum of them is taken as the number of units of terminal carload operation. This apportionment is based on handling by yard crews, but practically the same work is done at other points by road engines. The cost of maintenance of all tracks and equipment, all accounting work, and the various switching movements not directly assignable to any of the operations previously mentioned, are considered as spread uniformly over them.

In computing the number of these movements, the writer has considered interchange of traffic with the lines of the Grand Trunk Railway Company in the United States as not equivalent to terminal handling, but to movement through division points, which is a car-mile expense. The total tonnage handled is classified as follows, the number of cars being calculated at the ratio of 18.18 tons per car, the average loading of loaded cars at that time:

	Tons.	Cars.
Local Tonnage:		
Originated	6 732 000	370 400
Delivered to consignee	11 980 000	659 000
Interchange:		
Received from carriers	8 979 000	493 900
Delivered to carriers	4 886 000	268 700
Totals	32 577 000	1 792 000

Each car of local tonnage originated required the first three movements designated previously, or $3 \times 370\,400 = 1\,111\,200$ unit terminal movements. Each such car delivered required the fourth to sixth movements, or, $3 \times 659\,000 = 1\,977\,000$ units.

In interchange, the connecting road takes the place of the shipper or consignee, but there is no empty car to be placed or taken away. Each interchange, therefore, required two unit movements instead of three, making 1 525 200 units.

In 1913, empty car-miles amounted to 85 000 000. The average haul of loads was 178 miles, and this has been taken as the distance moved by each empty car. There were then 477 500 empty cars moved, each of which required the seventh and eighth movements, making 955 000 unit movements. The total was 5 568 400 unit movements.

The direct cost is shown by Table 8 to have been $3 901 252, or 70 cents per unit for operating expenses, and $1 387 850, or 25 cents per unit for interest, a total of 95 cents per unit of terminal handling.

Distance.—Train-Miles.—The operations of the Grand Trunk Railway Company for 1913 required 10 743 360 direct freight train-miles, at a cost for expenses peculiar to train movements of $2 520 920, or 23.5 cents per train-mile, practically all operating expenses.

Distance.—Car-Miles.—The freight car-miles amounted to 300 331 000, at a cost of $3 325 983, or 1.11 cents per car-mile, for operating expenses, and $834 100, or 0.28 cents for interest, a total of 1.39 cents per car-mile.

Distance.—Gross Ton-Miles.—There were 8 440 727 000 gross ton-miles produced at a cost of $817 200, or 9.7 cents per 1 000 ton-miles, practically all for operating expenses.

Resistance.—The unit of Resistance is that of lifting 1 ton 1 ft., or of 1 ft.-ton. The results of the experiments at the University of Illinois have been used in making velocity profiles. Calculations based on an assumed average weight of 35 tons per car, have been made showing the resistance necessary to be overcome in moving a train over each district of the System, making due allowance for acceleration, speed, grades, curvature, and stops, and tables have been prepared giving the foot-tons of work actually done in moving freight on the railroad during the period under consideration. The total resistance overcome was 275 454 000 000 ft.-tons. The cost was $5 820 060, or 2.11 cents per 1 000 ft.-tons for operating expenses, and $562 400, or 0.2 cents per 1 000 ft.-tons for interest, a total of 2.31 cents per 1 000 ft.-tons.

Indirect Cost.—About 8.5% of the direct cost is apportioned to terminal handling of less than carload freight. The same proportion of the indirect (freight) cost, or $830 000, therefore, is assigned to less than carload freight. About 25% of the direct cost is assigned to terminal carloads, which is independent of the distance freight is moved. The writer, therefore, apportioned 25% of the indirect cost, or $2 460 000, on the basis of tons handled terminally. This equals 7.5 cents per ton of carload freight. The remainder, $6 350 665, is apportioned on the basis of net ton-miles, amounting to 0.17 cent per ton-mile.

Similar analyses were made of the operations of the Grand Trunk Western Railway and the Atlantic and St. Lawrence Railway. The unit costs of all three roads were found to be as given in Table 9.

TABLE 9.

Railroad.	Cost of less than carload per ton, in cents.	Cost per carload per unit of operation, in cents.	Cost per train-mile, in cents.	Cost per car-mile, in cents.	Cost per 1 000 gross ton-miles, in cents.	Cost of resistance per 1 000 ft.-tons, in cents.
Grand Trunk.....	.....	0.95	23.5	1.39	9.7	2.31
Grand Trunk Western.	84.7	1.14	24.7	1.25	8.65	1.98
Atlantic and St. Louis.	80.0	0.72	28.1	1.24	12.00	2.40

The details making up each unit cost for the Grand Trunk Railway Company are given in Table 10.

Operating Conditions

The first step in the application of unit costs is the determination of the exact conditions of train and car loading under which traffic was moved. In applying the results of the calculations of resistance to cars of weights other

than the average of 35 tons, allowance must be made for the corresponding difference in frictional resistance. The correction does not apply on descending grades exceeding about 0.3%, as the resistance is then overcome by gravity.

TABLE 10.—DETAILS OF UNIT COSTS.

Account.	Terminal carload, in cents.	Train-mile, in cents.	Car-mile, in cents.	1 000 gross ton-miles in cents.	1 000 ft.-tons, in cents.
MAINTENANCE OF WAY:					
Track expenses	3.2	.....	.....	9.00	0.10
Buildings	1.3	0.14	0.021	.....	0.06
Miscellaneous expenses	.....	0.01	0.002	.....	0.01
MAINTENANCE OF EQUIPMENT:					
Locomotives, repairs	6.2	.....	0.082	.....	0.53
Freight cars, "	14.6	.....	0.408	.....	0.04
Miscellaneous expenses	1.3	.....	0.043	0.26	0.04
TRAFFIC	3.3	.....	.....	.....	.....
TRANSPORTATION:					
Dispatching	.....	0.68	.....	.....	.....
Station employees	3.5	1.00	0.031	.....	.....
" supplies and expenses	0.3	0.08	0.002	.....	.....
Yard expenses	22.2	.....	0.292	.....	.....
Joint yards and terminals	.....	.....	.....	.....	.....
Road enginemen	1.3	7.32	.....	.....	0.09
Engine house expense, road	0.4	.....	.....	.....	0.08
Fuel, road	4.3	.....	.....	.....	0.98
Water, etc., road	0.3	.....	.....	.....	0.06
Road trainmen	1.5	11.10	.....	.....	.....
Train supplies and expenses	.....	.....	0.105	.....	.....
Clearing wrecks	.....	0.36	0.009	.....	.....
Loss and damage, freight	1.4	0.90	0.022	.....	.....
Damage to stock	.....	0.08	.....	.....	.....
Miscellaneous expenses	2.2	0.82	0.052	.....	0.05
GENERAL EXPENSES	2.6	0.86	0.040	0.36	0.08
INTEREST:					
Freight locomotives	1.6	.....	0.021	.....	0.13
" cars	12.1	.....	0.150	.....	.....
Buildings	1.7	0.16	0.025	0.10	0.06
Yard and industrial tracks	5.0	.....	0.026	.....	.....
Hire of equipment	4.5	.....	0.055	.....	.....
Totals	94.8	23.51	1.386	9.72	2.31

Empty-Car Movement.—On all districts of the Grand Trunk Railway, there is throughout the year a preponderance of traffic eastbound. There is a large empty-car movement westward and a small one eastward. The cost of each belongs to the freight moved in the opposite direction. Where the traffic is of special character, requiring the return west, empty, of each car sent east, the calculation of cost is made accordingly. Otherwise, each loaded car east is charged with a fraction of an empty car west according to the ratio of empties to loads. The train-mile and car-mile costs of empty-car movement are the same as for loads. The gross ton-mile cost varies with the weight. The resistance cost is the same per foot-ton, but the resistance varies with the weight of the car.

Train Loading.—It is assumed that the ratio of car-miles to train-miles eastbound on each division represented the normal train loading for that division. The loaded car-miles westbound were divided by this ratio in order to

obtain the train-miles required to move west-bound freight. The remaining actual train-miles west were considered as belonging to the empty movement, and the number of empties per train was determined accordingly. It is true that loaded and empties moving westward are not separated in the manner described, but it appears to be improper to charge to the cost of west-bound freight any expense arising from its being mixed with empties hauled west in order to carry freight east.

Application of Unit Costs

The cost per car for a given freight movement is equal to the sum of the following:

The terminal carload cost per car, at each end of the movement.

The train-mile unit cost, multiplied by the distance moved, and divided by the number of cars per train.

The car-mile unit cost, multiplied by the distance moved.

The gross ton-mile unit cost, multiplied by the distance moved, and by the number of gross tons per car.

The resistance cost per foot-ton, multiplied by the number of foot-tons resistance per ton, and by the gross weight of the car, plus a proportionate part of the weight of the engine and the caboose.

Charge for empty-car movement in opposite direction, if any.

Table 11 shows the application of the unit costs to the movement of a car of dressed meats from Chicago to Portland, Me. The average load carried was 10 tons, with 3 tons of ice, making the gross weight 38 tons. The number of cars and of gross tons per train on each district were taken from tables compiled for the purpose from statistical records of the Grand Trunk Railway. The equivalent rise in feet, under resistance, was taken from velocity profiles, corrected for a 38-ton car.

The terminal cost at Elsdon, Ill., is the unit cost of the Grand Trunk Western Railway multiplied by 3, and covers the movements required to place the car in a train ready to leave the Elsdon Yard. This cost is assumed to be the same at all stations on the railroad. Probably some modifications should be made for special conditions at Elsdon, but this was not done, for the effect would be relatively small on so long a haul, and the exact conditions were not known. At Deering, Me., the rates of the Atlantic and St. Lawrence Railway Company were used.

The length of each district, divided by the number of cars per train, gives the number of train-miles, the cost of which must be borne by each car. The total tons under resistance equal the weight of the car, 38 tons, plus a proportionate part of the weight of the engine and tender, taken as 145 tons. Cars are handled from the stockyards to Elsdon, from Mimico, Ont., through Toronto, to York, Ont., Canada, and from Deering to Portland, Me., by yard engines, and the cost is included in yard expenses.

Certain corrections must be made if cars are owned by the shipper. Charges for freight-car repairs, train supplies, interest on freight cars, and

TABLE 11.—DIRECT COST OF TRANSPORTATION, CHICAGO, ILL., TO PORTLAND, ME., OF ONE CAR OF DRESSED MEATS, WEIGHING 10 TONS NET, 38 TONS GROSS.

District.	Miles.	Train.		Terminal Carload.		Train-Miles.			Car-Miles.			Gross Ton-Miles.			Resistance.					Total.
		Cars.	Gross tons.	Rate per unit.	Amount.	Number.	Rate, in cents.	Amount.	Number.	Rate, in cents.	Amount.	Number.	Rate per mile, in cents.	Amount.	Total tons.	Equivalent rise, in feet.	Foot-tons.	Rate per mile, in cents.	Amount.	
Grand Trunk Western Railway: Elsdon-Port Huron	324	34.5	1 044	$1.14	$3.42	9.4	24.7	$2.32	324	$1.25	$4.05	12 300	8.65	$1.07	43.3	6 250	271 000	1.98	$5.36	$16.22
Grand Trunk Railway:																				
Sarnia to Mimico	161	34.1	1 089	....	....	4.7	23.5	1.10	161	1.39	2.24	6 120	9.70	0.60	43.0	3 570	153 600	2.31	3.55	7.49
York to Turcot	325	39.7	1 228	....	....	8.2	23.5	1.93	325	1.39	4.52	12 350	9.70	1.20	42.5	7 500	318 500	2.31	7.35	15.00
Turcot to Norton Mills	132	31.5	1 060	....	....	4.2	23.5	.98	132	1.39	1.83	5 000	9.70	0.48	43.2	4 100	177 000	2.31	4.09	7.38
Atlantic and St. Lawrence Railway: Norton Mills to Deering	165	32.4	1 133	0.72	2.16	5.1	28.1	1.43	165	1.24	2.05	6 270	12.0	0.75	42.9	3 220	138 000	2.40	3.31	9.70
Totals	1 107	35.3	....	....	$5.58	....	....	$7.76	....	....	$14.69	.....	.....	$4.10	.....	.....	.....	.....	$23.66	$55.79

Deductions on account of privately owned cars	$1.95	$8.45	
Deductions on account of carriage as manifest freight		3.11	
Total deductions			$ 13.51
Additions: Rental for car, 1 121 miles at $0.01		$11.21	
Transfer through St. Clair Tunnel		1.00	
Total additions			12.21
Net total			$54.49

hire of equipment, as far as it has been applied to cars at an average rate, must be eliminated.

Another correction is to be made because dressed meats move in manifest, or fast freight, trains, which are made up so as to require less handling in yards than other trains; therefore, the charge for yard expenses, under car-mile costs, must be reduced for this class of business. The charge is estimated by the writer at one-fourth as much as for ordinary through freight, and the manifest is estimated as one-fourth the total through movement. Locomotive repairs would be similarly affected, as all this expense which is charged to car-mile costs is on account of through cars in yards. Of freight car repairs, about one-half come from the same cause, and would be affected in the same ratio. The results of these corrections are shown in Table 11.

Additions to the calculations mentioned are made because of rentals paid for the use of privately owned cars, 1 cent per mile, whether loaded or empty, and for passage through the St. Clair Tunnel, $1 for the loaded car and 50 cents for the empty.

An empty car hauled in the same train, would have one-third as much for terminal cost, and the same cost for train-miles and car-miles. The gross ton-miles would be proportional to the weight of the car. The "total tons", under resistance, would also be proportionately reduced, but the resistance per ton would be about 50% more for the light car. The total cost would be about 80% as much as for the load. The cost of the return of the empty refrigerator car to Chicago was calculated at $50.40, the more than average weight of the car and the charge for rent bringing the cost, in this case, to 90% of that of the load.

The full cost of the return must be charged to the east-bound freight because nothing was moving west for which this type of equipment was suitable. In a similar calculation for a commodity moving in general purpose cars, there should be included only that fraction of the cost of the empty car represented by the ratio of the empty cars west to the loads east. If, however, the question, instead of being one of distribution of costs among kinds of business already existing, was one of the cost of taking on additional traffic eastward, it would be proper to charge against it the entire cost of the empty-car movement west. On the other hand, additional traffic westward would not only have no empty movement against it, but the additional cost would be only the difference between the cost of moving a load and that of an empty, which would be small, as shown previously.

The indirect cost has not been computed for the Grand Trunk Western and the Atlantic and St. Lawrence Railways, owing to the difficulty of determining the investment in these subsidiary companies, but it may be taken as the same as that of the Grand Trunk Railway. The amount assigned to each ton of the freight under consideration would be:

1 ton at Elsdon, Ill.	$0.075
1121 ton-miles @ $0.017	1.900
1 ton at Deering, Me.	0.075
Total	$2.050

TABLE 11.—DIRECT COST OF TRANSPORTATION, CHICAGO, ILL., TO PORTLAND, ME., OF ONE CAR OF

The application of unit costs thus ascertained will always require care in allowing for the special conditions surrounding each case, but the writer believes that it will be found practicable to apportion costs of handling various commodities with an accuracy comparable to that of a manufacturer calculating the cost of the various products of his factory. Discussion is needed as to the principles set forth and the ratios used herein, in order that a wider range of experience and judgment may be brought to bear on them.

In the preparation of this analysis, other members of the Arbitration Staff of the Grand Trunk Railway Company were consulted repeatedly. Valuable assistance was rendered by Messrs. R. A. C. Henry, Engineer in Charge, and M. A. Fullington and F. W. Cooper, Division Engineers. Messrs. George O. Somers, Traffic Expert, and A. D. Porter, Mechanical Engineer, furnished much useful information. Mr. V. I. Smart and E. W. Moreland, Assoc. M. Am. Soc. C. E., who were assigned as Assistants on the work, were invaluable in assembling material and in the solution of the many problems which arose. The criticisms of J. G. Sullivan, M. Am. Soc. C. E., on the completed work caused much of it to be revised, and, it is believed, to be greatly improved.

The science of railroading covers too wide a field for any one man to be expert in every part of it. The writer was especially fortunate in having the advice and assistance of so many men of wide experience and ripe judgment. If the results prove useful in the solution of railroad problems, a large share of the credit will be due to these men.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

THE DESIGN OF STRUCTURAL SUPPORTS FOR TURBO-GENERATORS*

By Edward H. Cameron,† Assoc. M. Am. Soc. C. E.

Synopsis

The modern turbo-generator is a highly complicated machine, subject, at times, to intricate vibrations and to forces which may attain great magnitude, owing to the high operating speed. Its structural pedestal, of concrete or steel, must provide a substantial support, both during normal operation and at times of disturbance of normal conditions.

The design of such a pedestal presents many interesting problems to the civil engineer. The restraint of vibrations and the restriction of deflections to very small amounts are the controlling factors throughout the design.

In this paper, the relative merits of the three common types of pedestals, reinforced concrete, structural steel, and composite, are discussed. A brief outline is given of the nature of the vibrations, and the suggestion is made that a definite horizontal impact allowance should be adopted, in addition to the vertical impact allowance now generally used.

In conclusion, it is suggested that utility or fitness, rather than relative cost, should be made the sole basis of comparison in deciding on the type of pedestal to adopt for a turbo-generator installation.

When a reciprocating engine and generator is replaced by a modern turbo-generator, the operating engineer is always impressed with the marked economy of space and weight attained. The civil engineer who is called on to design a

* Presented at the meeting of December 20th, 1922. Correspondence is invited and may be sent by mail to the Secretary. Discussion on this paper will be closed with the April, 1923, *Proceedings*, and when finally closed, the paper, with discussion in full, will be published in *Transactions*.

† Civ. Engr., Jackson & Moreland, Boston, Mass.

structural support for the new unit, is equally impressed with the saving in weight and space, when he compares his foundation with that which previously served. In many instances, it is found practicable to utilize the sub-foundations of old engines for the support of modern units of many times the power of the machine replaced.

The introduction of the turbine-driven electric generator has caused the evolution of a distinct type of foundation. Where previously the support for the engine and generator consisted of a large sub-foundation and a massive superstructure, it is found possible, with the turbo-generator, to mount the unit on a comparatively light and open pedestal, with space in the basement below the machine for the condenser, circulating pump, and other auxiliaries. The absence of the unbalanced thrusts on the foundation, which occur with the reciprocating engine, has rendered this type of construction possible.

The fact that turbo-generators have only a fraction of the weight of reciprocating units of equal power, together with the freedom from the great impacts of the latter type, resulted, at first, in a tendency to slight the subject of turbine foundations. This is noticeable in some of the older reference books on power-plant design. It is now generally recognized that special problems occur in the design of such foundations, and that great care is necessary if an adequate and efficient support is to be obtained.

The generator with its prime mover is, perhaps, the most important element in the whole power plant, and principles of false economy should not be allowed to result in an inadequate supporting structure for it. The design should be made by an experienced engineer who realizes that it is much more than a problem of sustaining static loads, and who is familiar with the dynamic effects of the revolving machine during normal operation and at times of accident. This supporting structure may be considered as consisting of two elements, namely, the sub-foundation below the basement floor or condenser-pit level, which may be called the "mat" or "footing", and the steel or concrete structure extending from this level to the base of the machine, which is often referred to as the "pedestal."

The type of condenser selected for the unit is a controlling element in deciding on the type and proportions of the pedestal. The value of a high vacuum was early recognized in the development of the steam turbine and much study has been given to the construction of condensers to attain this end. At the same time, the ill effect of long piping and bends in the line between the condenser and the turbine exhaust has been emphasized, and a type of installation developed in which the condenser is mounted directly beneath the turbine with a resulting increase in efficiency. When financial and operating considerations result in the selection of a jet condenser for the unit, the problem of the design of the pedestal is much simplified, as there is generally ample space for the pedestal beams, columns, brackets, and bracing. The selection of a surface condenser, however, often makes necessary considerable ingenuity on the part of the structural designer to obtain a proper pedestal without interference of these members with the condensing unit.

Three types of pedestals have been evolved, as follows: (*a*) the all reinforced concrete type; (*b*) the structural steel type; and, (*c*) the combination structural steel and concrete, or composite, type.

The reinforced concrete pedestal is probably the most common type of turbo-generator support. The main advantage of this type lies in the relatively great volume of concrete in the beams and piers which have sufficient mass to absorb any vibrations that may develop in the machine, and, with reasonable care in design and construction, satisfactory operating conditions, as far as vibration in the pedestal is concerned, are practically assured. This pedestal has, however, several distinct disadvantages. The massive concrete members which are necessary to carry the loads, often seriously restrict the space required for the condenser, the ventilating air-ducts, and the piping. It is often quite difficult to secure as compact an arrangement for the condensing and auxiliary equipment as is desirable and, in general, the space below the unit for operation and dismantling will be more limited than with a steel pedestal. The weight of a concrete pedestal is several times as great as that of a steel pedestal. This results in a more expensive sub-foundation or mat, and may present a serious problem where subsoil conditions are poor.

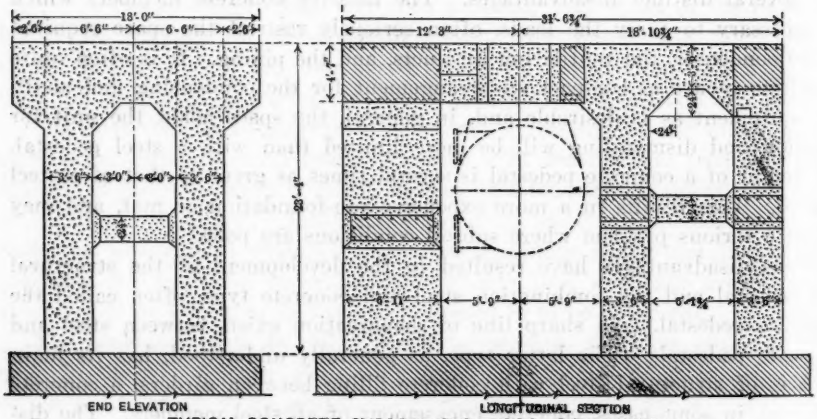
These disadvantages have resulted in the development of the structural steel pedestal and the combination steel and concrete type, often called the composite pedestal. No sharp line of demarcation exists between steel and composite pedestals. The latter type, as generally understood, has its main members of structural steel with concrete filling between girders, a concrete deck, and, in some cases, concrete encasement of all steel members. The disadvantages noted for the concrete pedestal are largely overcome if a steel or composite design is adopted, and, at points where clearances are most restricted, it is much easier to provide members of adequate strength.

It is often necessary to defer the completion of the erection of the pedestal until the condenser has been placed, due to the space needed for erecting the latter. In such cases, the construction schedule for the plant and equipment depends on the date of the erection of the condenser. Once the condenser is placed in position, it becomes desirable to complete the erection of the pedestal as soon as possible, in order to be ready for the generator. In such cases, the fact that less time is necessary to construct a steel pedestal than one of concrete, is an argument in favor of adopting the former type.

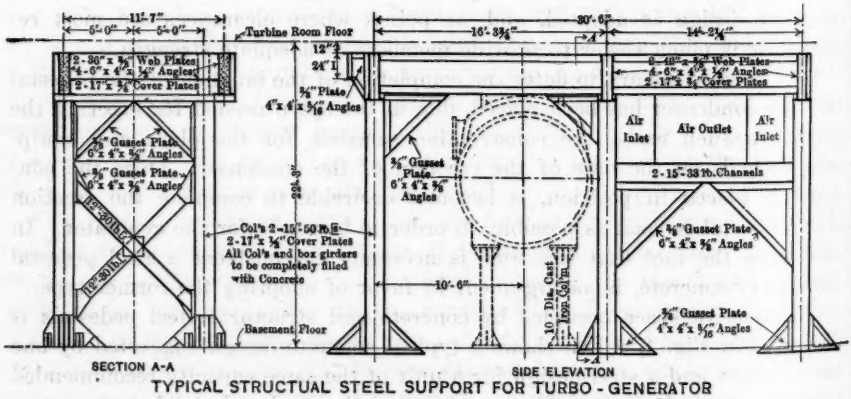
The relative space occupied by concrete and structural steel pedestals is illustrated in Fig. 1, which shows a typical concrete design suggested by one manufacturer and a steel design for a unit of the same capacity recommended by another manufacturer. The roominess of the steel pedestal is apparent.

The tendency of the steel pedestal to vibrate with the machine constitutes its one disadvantage when compared with one of concrete, and the design of a steel pedestal proportioned so as to avoid this vibration is a difficult problem. The tendency to vibrate exists in all turbo-generators. The unit is a complex machine operating at high speed, and vibratory forces and moments may exist, due to many causes. Normally, the vibrations are restrained to negligible amplitudes by care in the manufacture of the unit and by proper precautions on the part of the operator.

Vibrations may be due to excessive bearing clearances, deflection of the spindle, disturbance of clearances at various points producing rubbing, lack of alignment, or to unbalance in the rotor caused by erosion of blades, incrustations on blades, breaking of buckets, etc. Extreme care is taken by manufacturers to correct all unbalance in the machine, and it ordinarily leaves the shop in practically perfect balance. Assuming that it is properly installed, the principal factors which will tend to produce vibrations are those which may develop in the course of its continued operation in the form of one or another of the causes stated.



TYPICAL CONCRETE SUPPORT FOR TURBO-GENERATOR



TYPICAL STRUCTUAL STEEL SUPPORT FOR TURBO-GENERATOR

FIG. 1.

Unbalance of the rotor may occur in either or both of two forms. Static unbalance will exist when the center of gravity of the revolving parts does not lie in the geometric center of the shaft. Dynamic unbalance will occur if an unbalanced couple exists in any plane containing the shaft axis. Thus, a machine may be in static balance, but be unbalanced dynamically. The high speed of the turbine renders unbalance very serious as an unbalanced mass of 1 oz. will generate a centrifugal force of thousands of pounds.

Vibrations in the turbo-generator, from whatever cause, will be transmitted to a greater or less extent to the pedestal. If the latter is of a light, non-rigid construction, the vibrations will be amplified, resulting in serious damage to the unit. The pedestal must have mass enough to absorb all vibrations and be rigid enough to prevent swaying. If it is of great mass, the time necessary to overcome its inertia will be sufficient to consume the fractional part of a second elapsing until an impact in the opposite direction is applied, and its vibrations will be restrained to negligible amplitudes.

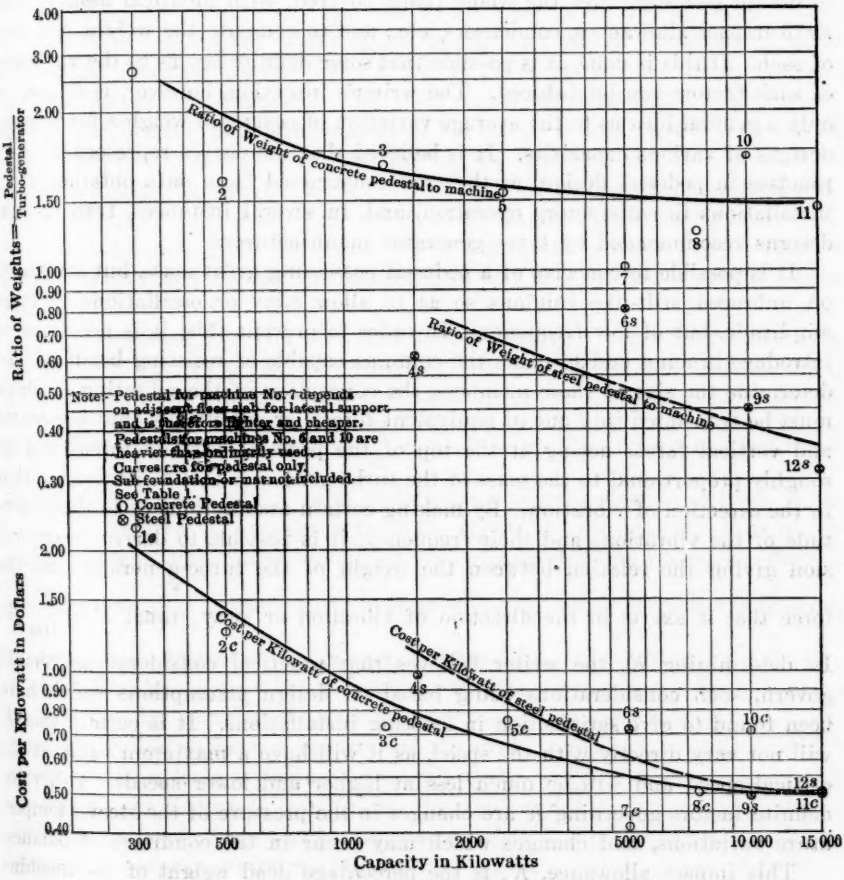

Ratio of Weights = Pedestal / Turbo-generator
Ratio of Weight of concrete pedestal to machine
Ratio of Weight of steel pedestal to machine
Note:- Pedestal for machine No. 7 depends on adjacent floor slab for lateral support and is therefore lighter and cheaper.
Pedestals for machines No. 6 and 10 are heavier than ordinarily used.
Curves are for pedestal only.
Sub-foundation or mat not included.
See Table 1.
○ Concrete Pedestal
⊗ Steel Pedestal
Cost per Kilowatt in Dollars
Cost per Kilowatt of concrete pedestal
Cost per Kilowatt of steel pedestal
Capacity in Kilowatts
300
500
1000
2000
5000
10 000
15 000

FIG. 2.

The most practical means of arriving at the requisite mass for a turbo-generator pedestal is by a study of existing designs in satisfactory operation, both as to the total mass and its distribution. In this connection, the weight-ratio curves of Fig. 2 are of interest. Each of the points plotted represents the ratio of the weight of the complete pedestal (excluding the sub-foundation or mat) to the weight of the unit supported, covering the range from 300 to 15 000 kw. Existing installations, and in some cases, typical manufacturers'

designs, have been used. The curves (Fig. 2), indicate that concrete designs, as now made, weigh from two to four times as much as steel designs. For both types, the weight ratio decreases as the units become larger.

It is not intended to convey the impression that the weight-ratio and cost curves shown in Fig. 2 will give the precise weight or cost of any pedestal that may be considered. The sources of information used are too diverse to permit of this, and the factors entering into the design are too variable to allow of such an interpretation. The only way to secure such curves would be to design pedestals over the whole range covered, with identical assumptions as to impact allowances, condensers, etc., and to compute the weight and cost of each. If this is done, it is possible that some definite law as to the variation of such factors can be deduced. The writer's intention, however, is to convey only a general idea as to the average variation of costs and weights for present designs of various capacities. It is believed that the curves represent average practice in pedestal design, as they are constructed from data obtained from installations in satisfactory operation, and, in several instances, from typical designs recommended by turbo-generator manufacturers.

It is possible to conceive of a pedestal possessing great mass, but supported on unbraced stilt-like columns so as to allow sway or oscillations of large amplitude, but of low frequency. In order to prevent this, it is necessary to introduce bracing and to make the columns capable of resisting bending. To determine the size of these members, the conception of the vibrating machine must be translated into one of equivalent forces, or specifically into horizontal and vertical forces acting at the top of the pedestal. These forces will be roughly proportional to the mass of the turbo-generator and to its acceleration in the direction of vibration. By making certain assumptions as to the amplitude of the vibrations and their frequency, it is possible to derive an expression giving the relation between the weight of the turbo-generator and the force that it exerts in the direction of vibration or sway, thus: $F = \frac{K}{100} W$.

In determining K, the writer believes that practical considerations should govern, such considerations being based on design assumptions which have been found to give satisfaction in existing installations. It is certain that K will not vary directly with the speed, as it will have a maximum value at the critical speed and will be much less at higher and lower speeds. Other indefinite factors governing K are changes in the pressure of the steam, temperature variations, and changes which may occur in the condition of balance.

This impact allowance, K, is the percentage dead weight of the machine acting as transverse horizontal and vertical forces at the top of the pedestal. There is a great diversity of opinion as to the proper impact allowances. This is due partly to the failure specifically to define the allowable unit stresses, as will be noted later, and often to ignoring the horizontal impact allowance altogether. In some cases, the practice seems to have been to adopt a high vertical impact allowance, together with small working stresses, and to ignore the horizontal forces which exist. It appears more logical to assume definite horizontal and vertical forces, in addition to the dead weight of the machine, and

to adopt working stresses more nearly equal to those used in general structural design. For ordinary designs, the writer has used horizontal and vertical impact allowances varying from 33⅓ to 50% of the dead weight of the machine.

Certain general principles exist relative to the proper proportions of the pedestal to resist best the horizontal impact force noted previously. These principles can best be considered by an examination of the pedestal frame as regards vibrations. Every pedestal has a "period of free vibration", that is, a definite time which would elapse during one complete oscillation, if it was sprung from its normal position and left free to oscillate without receiving further exterior impulses. This free period of the pedestal is dependent on the size, length, and arrangement of its members and on the material of which they are constructed. Complex computations would be necessary to determine just what would be the "free period" of any one type of steel pedestal, and uncertainties as to the effect of the joints would render such computations of little value. The more rigid a pedestal is made, the shorter its natural period will be, and its members should be arranged so as to secure this end.

Fig. 3 represents an end view of a typical pedestal. This frame will have the shortest natural period of vibration, and, therefore, maximum rigidity, when the center of gravity of the turbo-generator and pedestal is made as low as possible, and when the ratio of $\frac{H}{W}$ is made as small as clearances and available space will permit. In some installations, a low center of gravity is obtained by hanging the condenser from the top of the pedestal and thus utilizing its mass in addition to that of the turbo-generator and pedestal. Table 1 gives the ratio $\frac{H}{W}$, for several designs noted by the writer, together with other pedestal data. The machine numbers correspond to those used in constructing the curves of Fig 2.

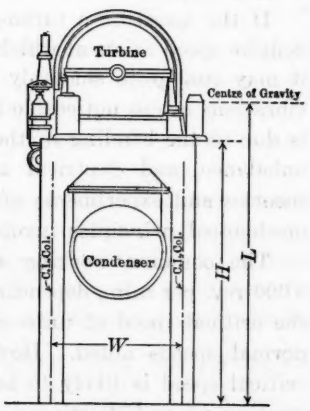


FIG. 3.

It should be stated that some designers consider it possible to have a pedestal of too great rigidity, although it is believed that all turbo-generator manufacturers concur in the desire that they be made very rigid. Comparison has been made between units resting on very rigid pedestals and instances of lathes and similar machines which have suffered from vibrations when supported directly on a concrete base. A design of a turbo-generator pedestal has been developed by Mr. N. W. Akimoff to overcome this objection of too great rigidity. In this design, the unit rests on a three-point support with heavy locomotive springs between it and the pedestal to absorb shocks. The writer understands that no pedestals of this design have been installed.

If the natural frequency of vibration of the pedestal or of any of its members equals the speed of the machine, resonance occurs. Successive impacts

will build up the kinetic energy of the vibrating pedestal, and the amplitude of the vibrations will increase until a balance is reached between the energy of each impact and that of the restraining forces, due to the stiffness of the pedestal, or of the member under consideration. Thus, resonance will result in high stresses in the pedestal.

TABLE 1.—TABULATION OF PEDESTAL DATA

Machine number.	Pedestal.	Capacity, in kilowatts.	Condenser.	Ratio of weight of pedestal to weight of turbo-generator.	PEDESTAL:				
					Weight per kilowatt, in pounds.	Cost per kilowatt.	H.	W.	$\frac{H}{W}$
1.....	Concrete	300	.....	3.17	195.0	$2.30	6.6	4.0	1.65
2.....	Concrete	500	Jet	1.69	105.4	1.25	10.0	4.25	2.35
3.....	Concrete	1 250	Surface	1.84	62.0	0.73	7.0	5.30	1.32
4.....	Steel	1 500	Surface	0.61	18.0	0.99	17.75	8.33	2.13
5.....	Concrete	2 500	Surface	1.58	48.6	0.76	17.5	6.46	2.71
6.....	Steel	5 000	Surface	0.82	22.5	0.72	18.0	9.5	1.90
7.....	Concrete	5 000	Jet	1.02	27.9	0.41	19.21	8.29	2.35
8.....	Concrete	7 500	Jet	1.26	33.5	0.50	23.30	8.0	2.92
9.....	Steel	10 000	Surface	0.46	13.9	0.49	22.25	11.58	1.92
								10.0	2.23
10.....	Concrete	10 000	Surface	1.91	57.4	0.71	23.33	9.5	2.45
11.....	Concrete	15 000	Jet	1.45	44.2	0.49	20.0	10.5	1.9
								12.0	1.67
12.....	Steel	15 000	Surface	0.32	9.3	0.51	20 0	18.0	1.17

A quantitative determination of vibrations induced in a turbo-generator due to non-rigidity of the pedestal has been noted.* In this instance, serious vertical vibrations in the generator bearing of a 10 000-kw. unit were detected and measured by a vibrograph. The supporting beam under the bearing and its columns were reinforced, and it was found that the vibrations were reduced to safe amplitudes.

If the speed of a turbo-generator is varied, it is observed that a certain definite speed exists at which vibrations in the unit are quite marked, although it may run quite smoothly at all other velocities. This speed at which the vibrations are so noticeable is called the "critical velocity." This phenomenon is due to the bending of the shaft under the centrifugal forces generated by unbalance, and electrical and mechanical engineers are familiar with the theories and experiments of DeLaval, Stodola, and others, demonstrating the mechanical principles involved.

The common working speeds of turbo-generators are 1 500, 1 800, and 3 600 rev. per min., depending on the cycle and number of poles. In general, the critical speed of units as now constructed is greater than the two lower normal speeds noted. However, for machines of 3 600 rev. per min., the critical speed is likely to be below the working speed, and in such cases, in starting up and shutting down, it must be passed through. Although manufacturers make special provisions in machines operating above the critical speed, it is well for the pedestal designer to make a more generous impact allowance for high-speed machines.

* *Mechanical Engineering*, August, 1922.

In addition to the transverse horizontal forces which have been considered, vibrations and impacts may exist in a direction parallel to the axis of the shaft. The unit is self-contained as far as axial forces due to unbalanced steam flow is concerned, as the thrust-bearings transmit these forces to the casing where they are balanced by the steam pressure in the opposite direction. Vibratory vertical impacts, however, will produce axial impacts if any eccentricity exists between a bearing and the pedestal member which supports it, and, in general, it is found necessary to provide longitudinal bracing as substantial as the transverse bracing.

Another source of shocks which may come on the pedestal, lies in the disturbance of the magnetic torque of the generator, which takes place during short circuits. Under normal operating conditions, the magnetic torque need not be considered, as far as the pedestal is concerned, but short circuits will cause violent fluctuations of this torque, and the pedestal will receive serious impacts.

TABLE 2.

Force.	Direction.	Point of application.
Weight of generator	Vertical	Load points given on manufacturers' plans.
" " turbine	"	
" " bed-plate	"	
Dead weight of members	"	
Vertical impact allowance	"	
Transverse impact allowance	Horizontal	At center line of shaft.
Condenser and water content	Vertical	Dependent on method of carrying condenser.
Vacuum load	"	
Axial forces	Horizontal	At center line of shaft.

The following steps to be pursued in the design of a pedestal, are taken from standards used by the writer in the structural designs of power plants. No specific directions are attempted, as there is such a wide variation in loads, load points, construction of units, etc., for turbo-generators of different capacities and manufacture, but certain general steps are outlined for the guidance of structural designers engaged in the work, as follows:

1.—Decide on the probable type to be used: Concrete, structural steel, or composite.

2.—Preliminary layout, working from certified prints of equipment, piping, and electrical plans, to determine the best arrangement for beams, columns, and bracing, using assumed sizes. At this time, the piping and electrical layouts should be advanced sufficiently to indicate the necessary clearances for all elements adjacent to the pedestal. Investigate the approximate limits for H and $\frac{H}{W}$. H will be determined by the condenser size, floor levels, and crane clearance.

3.—Design the pedestal members in the following order, in which a 5 000-kw., Westinghouse, semi-double flow unit has been taken as an example:

(*a*) Design the slab and secondary members at the top of the pedestal.

(*b*) Compute R_1, R_2, R_3, M_1, M_2, M_3, and the shears for the main longitudinal girders; compute the average of the bending moment curves, *A* and *B;* Columns 1 and 3 (Fig. 4), to take the moments, $\frac{M_1}{2}$ and $\frac{M_3}{2}$, respectively.

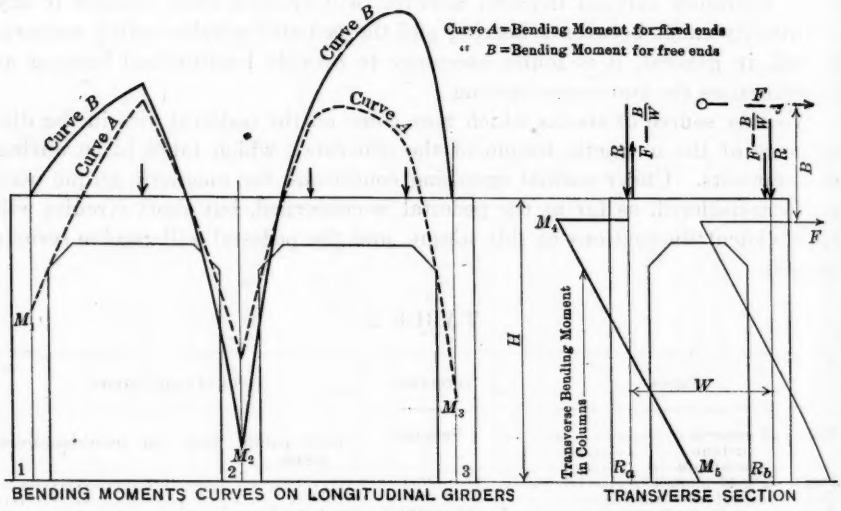


FIG. 4.

For a steel design, select by trial a beam to give a deflection, in either span, of less than 0.02 in. For a tentative value, let $f = 5\,000$ lb. per sq. in.

For a concrete design, select a beam for bending, $f_c = 600$ lb. per sq. in. Compute the deflection, maximum $d = 0.02$ in.

(*c*) Investigate the transverse bents, Nos. 1, 2, and 3, in Fig. 4:

F = horizontal impact assumed to act at the center line of the shaft.

R = beam reactions plus any loads from the unit that are directly over the columns.

Replace F by the two forces, $F \times \frac{B}{W}$, and an equal force, F, acting at the top of the pedestal.

Compute the reactions, R_a and R_b, and the moments, M_4 and M_5.

Assume the column size and check for the fiber stress (*f*).

Let P = the maximum vertical load on column in question.

$$\text{For Bent (1)}, f = \frac{P}{A} + \frac{M_4}{S_{xx}} + \frac{M_1}{2S_{yy}}$$

$$\text{" " (2)}, f = \frac{P}{A} + \frac{M_4}{S_{xx}}$$

$$\text{" " (3)}, f = \frac{P}{A} + \frac{M_4}{S_{xx}} + \frac{M_3}{2S_{yy}}$$

(*d*) Design the bracket, diaphragm, or braces in the transverse bent to take the direct stress or moment, or both, according to the arrangement. Design the column bases, sub-foundation mat, and remaining elements.

4.—Provision for condenser and vacuum loading: If the condenser is connected to the turbine by an expansion joint and the weight is carried from the top of the pedestal, the weight of the condenser and water content, together with the vacuum load, must be included in computing the bending moment curves. If the condenser rests on beams framed into the columns, the bending moment diagrams for the columns shown must be modified. If the condenser is rigidly connected to the turbine, and supported on springs which are independent of the pedestal, add the load on the turbine joint allowed by the manufacturers to the other forces acting at the top of the pedestal.

5.—To care for axial forces, provide longitudinal brackets or bracing equivalent to the transverse bracing.

6.—Compute the weight of the complete pedestal and compare it with the typical weight curves.

A wide diversity exists as to proper impact allowances and working stresses, as will be noted in Table 3 which gives stresses recommended by various authorities for structural steel designs.

TABLE 3.

Reference consulted.	Stress, in pounds per square inch.	Impact assumption.	Stress, in pounds per square inch, reduced to condition of dead load without impact.
1.....	One-third the ordinary.	.....	5 333*
2.....	6 000	"To take care of all loading, including impact and vibration."	6 000
3.....	10 000	"For the total load, to which has been added 25% of the machine loading to cover impact."	8 000
4.....	16 000	100% impact for all steel work.....	8 000
5.....	Halve the allowable stress and double the calculated stress.	.....	4 000*

* Based on a standard working stress of 16 000 lb. per sq. in.

As stated previously, it is believed that this diversity of opinion is due to the failure to consider the horizontal forces to be sustained. In pedestal designs, the writer endeavors to make conservative assumptions as to all horizontal forces to be considered as well as vertical dead and impact loads, and to design for these forces, using working stresses that are standard for general structural designs. This practice has resulted in designs that have proved adequate in every respect. For steel members in bending, the deflection limit will result in the selection of sizes that will give stresses that are much lower than standard.

Probably in few cases of structural design, must deflections be kept within such small limits, a common value for the maximum allowable deflection being 0.02 in. A continuous beam with fixed supports will deflect least under a given load, and the longitudinal girders should be designed to secure this condition.

These girders should be made continuous over the center columns. Absolute fixity of the end supports will not exist as some deflection at these points will occur, but the true bending moment curve may be approximated by averaging the ordinates of the "free end" curve and the "fixed end" curve (see the bending moment curves, *A* and *B*, Fig. 4). Columns 1 and 3, Fig. 4, must be designed for the bending moments, $\frac{M_1}{2}$ and $\frac{M_3}{2}$, respectively, in addition to their axial loads.

In the design of connections, gussets, and other details, assumptions should be conservative throughout, bearing in mind that vibrations should be restrained in all directions. Columns should be of ample section with relatively short unsupported lengths, and in order to secure this condition, brackets should be made as deep as possible at the column heads, and it is sometimes advisable to provide transverse brackets at the column bases. Structural sections less than $\frac{5}{16}$ in. thick should be avoided. The use of long cross-bracings has been criticized as they tend to allow sway before they come into operation.

Good results have been obtained with designs in which the top of the pedestal is of open steel construction, with steel plates or gratings forming a floor at this level. In such designs, it is desirable to insulate the bearing surfaces carrying the plates or gratings, with cork or felt, in order to eliminate vibrations of an annoying nature in the floor. A better design, however, is furnished by making the floor in the form of a thick reinforced concrete slab. This not only produces a vibration-absorbing mat at the most effective location, but also provides a good insurance against horizontal disturbance of the alignment of the unit. If a concrete floor is used, ample temperature steel should be provided, as the temperature range is greater than that under ordinary conditions and cracks are likely to develop.

The close proximity of the live steam lead to the exhaust outlet for some types of turbo-generators often makes it difficult to insert a longitudinal beam of adequate size in this space. In the steel design shown in Fig. 1, the longitudinal girder at the turbine end has been located outside the live steam lead, thus avoiding this difficulty. This design would be improved by making the longitudinal girders continuous over the center columns. The concrete filling between the webs of the longitudinal girders shown is a common means often adopted to secure additional rigidity and mass.

Operating engineers have observed that sudden changes in the temperature of the basement may affect the even running of the turbo-generator, due to the unequal expansion of the steel columns of the pedestal. To avoid this difficulty, these members are sometimes encased in concrete to insulate them from sudden temperature changes. Some designers consider it advisable to encase all structural steel members in concrete for fire protection and to obtain additional mass. Fig. 5 illustrates the structural steel pedestal for one of the 30 000-kw. units at the L Street Station of The Edison Electric Illuminating Company of Boston, Mass., and Fig. 6 is a photograph of the same unit in operation, with the steel pedestal encased in concrete.

The sub-foundation below the basement floor level has been referred to as the footing or mat. In its design, the disastrous effects of settlement on the

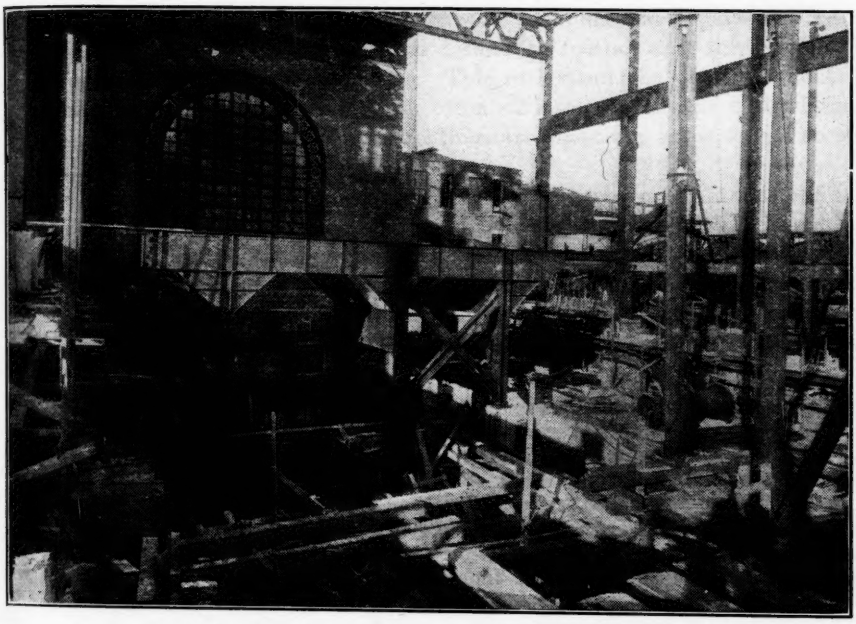


FIG. 5.—STEEL SKELETON FOR PEDESTAL FOR 30 000-KW. TURBO-GENERATOR.

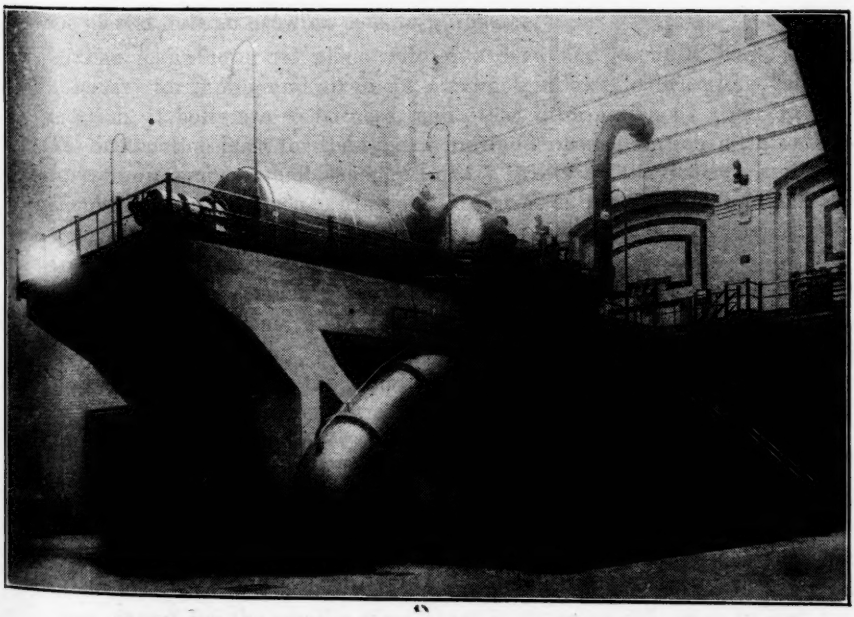


FIG. 6.—COMPLETED PEDESTAL FOR 30 000-KW. TURBO-GENERATOR.

alignment should constantly be borne in mind. In some designs, separate footings have been used for the individual columns and satisfactory results obtained, but it is better practice to design the footing as a single mat large enough to include all the columns. This mat should be of ample thickness to prevent local settlement, and it is often advisable to provide steel reinforcement to secure this end. Column footings are sometimes made to rest on steel grillages encased in the mat.

Certain details are common to practically all turbo-generator installations, and the designer of the pedestal should familiarize himself with them before starting his design. The important question of clearances for the condenser, auxiliaries, and their piping, should be borne in mind, and certified prints of the apparatus should be obtained at the start so that space for operation and dismantling may be assured. At this point, a turbo-generator manufacturer may be quoted as follows:

"The foundation must not extend inside the bed-plate facing where it may interfere with the ventilating ducts under the generator, and provision must be made for these ducts communicating with the cooling air. Provision must also be made for bringing out leads under the bed-plate and through the foundations if necessary.

"Foundations must be designed with the idea constantly in mind of the complete installation of the unit and possible future repairs. Provisions must be made for the removal and replacement of parts such as oil cooler, pump, steam chest, throttle and inlet, exhaust and bleeder piping connections, and supports, and also exhaust connections, expansion joints, and atmospheric outlet piping. This means that the question of interference must be carefully gone into in connection with all parts approaching the plane of the lower surface of the bed-plate and extending below the same. The positions of the centers of loading and magnitudes and the location of loads are given on the official outline drawing sent to purchaser."

The manufacturers' plans will give dead and vacuum loads, clearances necessary for piping and air-ducts, maximum allowable deflections, anchor-bolt location, if bolts are to be used, and other information of a general nature. The instruction book for the type of unit under consideration will also be found to contain many valuable suggestions. Prints of the footing and pedestal should be sent to the manufacturers as soon as practicable for their suggestions, so that their final comments may be obtained before letting the contract for the work.

If a surface condenser is to be used, a trolley beam should be provided to facilitate the removal of the condenser head. If the generator is to be bolted down, provision should be made for removing the bolts from below in order to avoid raising the unit for dismantling. To avoid the transmission of vibrations, it is generally considered advisable to isolate the pedestal and footing from the remainder of the power plant.

Curves are shown in Fig. 2 giving the estimated costs per kilowatt of pedestals (excluding the sub-foundation or mat) for several existing installations, and for certain typical designs suggested by manufacturers of turbo-generators. Present-day prices have been used, and percentage charges, such as profit, overhead, bond, and liability insurance, are included. It will be noted that, except for the smaller sizes, there is no great difference in cost between

the concrete pedestal and the steel pedestal. For ordinary designs, the cost of the pedestal should not exceed about 5% that of the turbo-generator, and, in general, any saving of one particular type of pedestal over another will not be more than about 1% of the cost of the unit which it supports. Considering the cost of the turbo-generator, and the important part in its satisfactory operation that its supporting pedestal plays as to freedom from vibrations and accessibility, it is believed that the latter's adaptability to the operation of the unit should be the only consideration in deciding which type of pedestal to adopt. Each type has its field. For the smaller units, it is believed that more concrete pedestals are in use, whereas, the reverse is probably true of the larger units. The composite type, combining the advantages of both concrete and structural steel, is coming to be more and more favored for all sizes of turbo-generating units.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

SPECIAL COMMITTEE ON SPECIFICATIONS FOR BRIDGE DESIGN AND CONSTRUCTION

FINAL REPORT ON SPECIFICATIONS FOR DESIGN AND CONSTRUCTION OF STEEL RAILWAY BRIDGE SUPERSTRUCTURE*

COMMITTEE

HENRY B. SEAMAN, *Chairman,*

J. H. AMES,
V. H. COCHRANE,
J. E. GREINER,
O. E. HOVEY,
C. W. HUDSON,
E. F. KELLEY,
M. S. KETCHUM,
S. B. BLACK,
I. F. STERN,
F. E. TURNEAURE,

C. R. HARDING, *Secretary.*

November 18th, 1922.

* To be presented to the Annual Meeting, January 17th, 1923.

To the President and Board of Direction,

American Society of Civil Engineers:

The Special Committee on Specifications for Bridge Design and Construction has held three meetings since its report* of November 16th, 1921, was presented to the Annual Meeting of the Society on January 18th, 1922, and has completed its Specifications for Steel Railway Bridge Superstructure.

The Tentative Specifications for Steel Railway Bridges which were submitted with the report mentioned, have been fully discussed,† and the specifications presented herewith are the result of the Committee's interpretation of such discussions.

It seemed advisable that these specifications should not follow the exact lines of specifications previously offered by other technical societies, as the purpose of this Society is advisory rather than administrative. For that reason, the subject is treated in a general manner, instead of entering on explicit details which should be left to the individual engineer in the execution of work under his direction. The Committee has reviewed the subject again from its origin, has considered all suggestions offered, and, in formulating these final specifications, has endeavored to cover the fundamental principles of design, leaving to each engineer the privilege of adding such details as may particularly meet the condition of his practice, or the requirements of his railroad.

Although the specifications follow what, for many years, has been considered good practice, they also recognize the natural progress of more recent experience and study. They represent, it is believed, the most advanced practice that is justified at the present time.

It is considered desirable that a specification emanating from this Society should apply to spans of indefinite length, that such details as the number of trusses to be used, and other matters of varied opinion, should be left to individual judgment, as influenced by local or special conditions, and even in a matter such as the clearance diagram, it is suggestive rather than final.

For estimating the dead load, the weights of gravel and ballast have been classed together as 120 lb. per cu. ft., and all timber has been specified as 5 lb. per ft. B. M., as treated timber may be introduced at any time during the life of the bridge. The weight of steel rails, with fastenings, has been specified as 200 lb. per lin. ft. of track, as suggested in the discussion.

Perhaps the greatest advance has been made in the provision for live load. The Class M engine, proposed by D. B. Steinman, M. Am. Soc. C. E.,‡ has so many advantages for heavy loading that the way has been opened for its gradual introduction, if desired. Suggestions that similar results might be obtained by using heavy Class E engines, with relatively lighter train loads, do not conform to the present tendency toward heavier cars. The typical engines, E-10 and M-10, have been used as a basis, and moment diagrams have been inserted in Appendix I, as from these, any proportional increase may be readily produced.

---

* *Proceedings*, Am. Soc. C. E., December, 1921, p. 683.

† *Proceedings*, Am. Soc. C. E., December, 1921, April, May, August, and October, 1922.

‡ *Proceedings*, Am. Soc. C. E., May, 1922, p. 1043.

Much attention has been given by the Committee to the subject of impact. Various formulas were studied and compared with the tests, but none was found to conform with equal satisfaction to the tests of chord and of web members. It was thought that the information may not yet be sufficiently complete to justify a change from the formula which has been in service many years and has produced serviceable bridges.

It has been well recognized by those who have the responsibility of the maintenance of old bridges that they should be properly proportioned for the ultimate load, as an overloaded bridge will not receive equal service by any other method; some members of the truss might be relatively weak and require the condemnation of the entire bridge. As a step toward meeting this condition, Article 106, on overload, has been inserted. There is great reluctance in adopting the overload stress of 24 000 lb. per sq. in., but it is believed that when the stress of 20 000 lb. per sq. in. is exceeded, a "slow order" will be placed on the bridge, and the structure watched for signs of deterioration. It is recognized, however, that the bridge should be proportioned for extreme contingencies.

In the matter of longitudinal force, attention is called to the error of past practice, which has been honored more in its breach than in its observance. The coefficient of friction, estimated at 20%, has been the basis of all specifications for the sudden starting or stopping of trains; but the fact that the brakes on freight cars are designed so that the wheels will not skid under an empty car, limits their efficiency to 20% of that weight, and as a loaded car may weigh four times as much as an empty one, the braking power, being 20% of the empty car, will be only 5% of the loaded car for which the bridge is designed. The specification of 10% for sliding friction of the train is, therefore, conservative.

After a study of the full-sized column tests and the derivation of various column formulas, keeping in mind the requirements of long-span bridges and the rating of existing bridges, the Committee adopted the Euler-Rankine column formula, $p = \dfrac{16.0\ k}{1 + \dfrac{l^2}{13\ 500\ r^2}}$, for design, believing it to be the most practicable formula of scientific origin, long established in practice, and best applicable to unusual or extraneous conditions. The use of the right-line formula, $p = 16.0\ k - 70\ \dfrac{l}{r}$, would be permitted under these specifications, but if this right-line formula had been specified, it would have precluded the use of the Euler-Rankine formula with its many advantages.

Although the derivation of the Euler-Rankine formula makes it applicable to columns of indefinite length, it has been thought that, for vibrating structures, the length of the column should be limited so as to insure rigidity under sudden motion. The assumed limits placed in Article 202 are for this purpose, and may be modified later, if experience or judgment so dictates.

In providing for the increased allowable stress on steel of greater strength than that of the structural grade, it is recognized that the modulus of elasticity

of the stronger steel is practically unchanged, and the Euler-Rankine formula, with a modified numerator and denominator, as specified, is equally applicable. For the stronger steel, in which the yield point may approach its ultimate strength, the proportional increase of the allowable stress may be so close to the ultimate as to encroach on a proper factor of safety. For this reason, the increase has been restricted in application to a yield point of not more than 70% of the ultimate strength.

In his discussion* of the strength of lattice-bars, Otis E. Hovey, M. Am. Soc. C. E., assumed that the curve of flexure of the column was a parabola and determined the numerator of his formula to be $4\ P\,l$. If the curve had been assumed to be that of a sinusoid, the numerator would have been $\pi\ P\,l$. It may be assumed, therefore, that an intermediate value of $3.357\ P\,l$ may be used for the numerator and thus simplify the formula to $R = \frac{P\,l}{4\,000\,y}$. This formula has been adopted by the Committee.

The spacing of stiffeners on plate girders does not justify undue refinement, and it is believed that the simple clause adopted will meet every purpose.

The specification for workmanship has been condensed as much as seemed to be practicable to meet the requirements of first-class work. The requirement for sub-punching and reaming has been generally avoided in order to save unnecessary expense, but it has been found that even the accuracy of the multiple punch will not produce a large number of plates that may be satisfactorily assembled without reaming. When the plates exceed five in number, it is deemed advisable to sub-punch and ream.

Heretofore, the permissible difference between pin-hole and pin, 5 in. in diameter or less, has been restricted to $\frac{1}{32}$ in. This restriction has been extended to apply to pins 8 in. in diameter, or less.

The Committee presents the Specification for Steel Railway Bridge Superstructure in completed form, and recommends its adoption.

Respectfully submitted,

For the Committee,

HENRY B. SEAMAN,
Chairman,

C. R. HARDING,
Secretary.

COMMITTEE:

J. H. AMES,
V. H. COCHRANE,
J. E. GREINER,
C. R. HARDING,
O. E. HOVEY,
C. W. HUDSON,
E. F. KELLEY,
M. S. KETCHUM,
H. B. SEAMAN,
S. B. SLACK,
I. F. STERN,
F. E. TURNEAURE.

* *Proceedings*, Am. Soc. C. E., April, 1922, p. 922.

CONTENTS

SECTION.		ARTICLE.
A.	General	1– 12
1.	Loads and Stresses	101–115
2.	Unit Stresses	201–204
3.	Details of Design	301–335
4.	Workmanship	401–431
5.	Full-Size Eye-Bar Tests	501–506
6.	Weighing and Shipping	601–604
7.	Structural Steel for Bridges	701–716
8.	Structural Nickel Steel	801–819
	Appendix.	

SPECIFICATIONS FOR STEEL RAILWAY BRIDGE SUPERSTRUCTURE

SECTION A.—GENERAL

1.—These specifications cover the design of fixed span bridges of all lengths and require first-class design, material, and workmanship.

2.—Material.—Bridges shall be made wholly of structural steel, except where otherwise specified. Castings shall be of steel, unless cast iron is specifically authorized by the Engineer.

3.—Notice to Engineer.—The Contractor shall give ample notice to the Engineer of the beginning of rolling and of shop work, so that inspectors may be provided. No material shall be rolled, or work done, before the Engineer has been notified where the orders have been placed.

4.—Type of Bridge.—The type of bridge to be used for various span lengths may be as follows:

Rolled beams up to.....	30 ft.
Plate girders from.....	30 to 125 ft.
Riveted trusses from.....	100 ft. and over
Pin-connected trusses from.....	150 ft. and over

5.—Spacing of Girders and Trusses.—The girders of deck bridges and stringers, where two are used on each track to carry open floors, shall not be spaced closer than 6 ft. 6 in., center to center.

The width between centers of trusses or girders shall be sufficient to give lateral stiffness and to prevent overturning by the specified lateral forces, and, in no case, shall be less than one-twentieth of the span.

6.—Depth Ratios.—The depth of trusses preferably shall be not less than one-tenth of the span. The depth of plate girders preferably shall be not less than one-twelfth of the span. The depth of rolled beams used as girders and the depth of solid floors preferably shall be not less than one-fifteenth of the span. If less depths are used, the section must be increased so that the maximum deflection will not be greater than if these limiting ratios had not been exceeded.

7.—Clearance.—The clearance on straight track shall not be less than that shown in Fig. 1. On curves, additional provision shall be made for a car, 80 ft. long, 60 ft. between truck centers, 14 ft. high above the top of a 6-in. rail, and with allowance for super-elevation of the outer rail. Unless otherwise specified, the super-elevation of the outer rail shall be ¾ in. for each degree of curvature, with a maximum of 6 in.

8.—Skew Bridges.—In skew bridges without ballasted floors, the end stringers or end girders for each track shall be square to the line of track.

9.—Timber Floor.—Ties shall not be less than 10 ft. long, spaced about 6 in. apart, and secured against bunching. If subjected to bending, the stress in the extreme fibers of ties shall not exceed the unit stress for timber given in Article 203, assuming a maximum wheel load, with 100% impact, to be distributed over three ties.

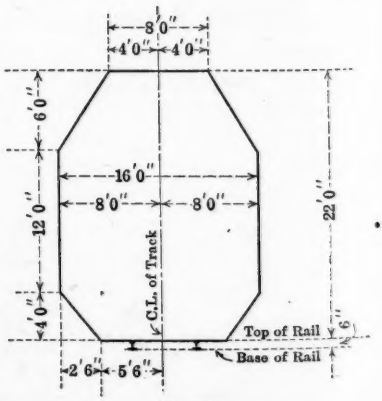


FIG. 1.

10.—Ballasted Floors.—Ballasted floors shall have at least 6 in. of ballast under the ties, and the ballast shall

be assumed as level with the base of rail, the weight of the ties being neglected. The live load on each tie shall be assumed as one-third of the maximum axle load distributed over a width of 10 ft.

11.—Dimensions for Calculations.—For the calculation of stresses, the length shall be:

For trusses and girders, the distance between centers of bearing;
For floor-beams, the distance between centers of trusses;
For stringers, the distance between centers of floor-beams;

and the depth shall be:

For pin-connected trusses, the distance between centers of pins;
For riveted trusses, the distance between the centers of gravity of the chords;
For plate girders, the distance between the centers of gravity of the flanges (not to exceed out to out of flange angles) unless net section modulus is used.

12.—Spans with floor systems shall preferably have end floor-beams.

Section 1.—Loads and Stresses

101.—Loads.—Stresses shall be shown separately for the following: Dead load, live load, impact, centrifugal force, and lateral and longitudinal forces. Members shall be proportioned for the combination giving maximum sections, except as otherwise provided.

102.—Dead Load.—In estimating the weight of the structure, for the purpose of computing dead load stresses therein, the following unit weights shall be used:

Steel	490	lb. per cu. ft.
Concrete	150	" " " "
Sand, gravel, and ballast	120	" " " "
Asphalt-mastic and bituminous macadam	150	" " " "
Granite	170	" " " "
Paving bricks	150	" " " "
Timber	5	" per ft. B. M.

The rails, inside guard rails, and fastenings shall be assumed to weigh 200 lb. per lin. ft. for each track.

103.—Live Loads.—The live load for each track shall consist of typical engines followed by a uniform train load, according to either Class E series, or Class M series, as may be specified by the Engineer. It shall be a multiple of one or the other of the loads with wheel spacings, as shown in Figs. 2 and 3.

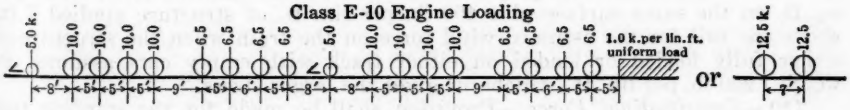


FIG. 2.

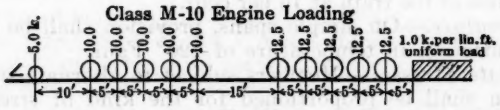


FIG. 3.

Loading E-60 or Loading M-50 is recommended for main-line bridges of American railways.

104.—Impact.—The dynamic increment of the live load shall be added to the maximum computed live load stresses and shall be determined by the formula:

$$I = S \frac{300}{L + 300},$$

in which,

I = impact or dynamic increment to be added to live load stresses;
S = computed maximum live load stress;
L = loaded length of track, in feet, producing the maximum stress in the member. For bridges carrying more than one track, the aggregate length of all tracks producing the stresses shall be used.

Impact shall not be added to stresses produced by longitudinal and lateral or wind forces.

105.—For bridges designed exclusively for electric traction, impact shall be taken as one-third of that given by the impact formula.

106.—Allowance for Increase of Live Load.—Wherever an additional counter would be required, or reversal of stress would be caused, or heavier dimensioning would result, a member and its details shall be designed for a live load 50% greater than that given in Article 103, with an allowance of 50% increase in unit stresses.

107.—Centrifugal Force.—Structures on curves shall be designed to resist the centrifugal force of the live load applied 7 ft. above base of rail, as computed by the following formula:

$$C = \frac{0.067\ W\ V^2}{R}$$

in which,

C = horizontal centrifugal force;
W = live load, including impact;
R = radius of curve (5 730 ÷ degree of curve);
V = speed, in miles per hour = (60 — 2½ times the degree of curve).

108.—Lateral Forces.—All spans shall be designed for a lateral force on the loaded chord of 200 lb. per lin. ft. plus 10% of the specified train load on one track, and 200 lb. per lin. ft. on the unloaded chord, these forces being considered as moving.

109.—Wind Force.—Viaduct towers shall be designed for the one of the following loads that causes the greater stress:

Viaduct towers shall be designed for a force of 50 lb. per sq. ft. on one and one-half times the vertical projection of the structure unloaded; or 30 lb. per sq. ft. on the same surface plus 400 lb. per lin. ft. of structure applied 7 ft. above the rail for the assumed wind force on the train when the structure is either fully loaded or loaded on either track with empty cars assumed to weigh 1 200 lb. per lin. ft., whichever gives the larger stress.

110.—Longitudinal Force.—Provision shall be made for the starting and stopping of trains, with a coefficient of friction on the engine drivers at 20% and on the balance of the train at 10 per cent.

111.—Temperature.—On simple spans, provision shall be made for expansion due to a variation in temperature of 120° Fahr.

112.—Alternate Stresses.—Members subject to alternate stresses of tension and compression shall be proportioned for the kind of stress requiring the larger section. If the alternate stresses occur in succession during the passage of one train, as in stiff counters, each stress shall be increased by 50% of the smaller. The connections of such members shall be proportioned in all cases for the sum of the stresses thus increased.

If the live load and dead load stresses are opposite in character, only two-thirds of the dead load stress shall be considered as effective in counteracting the live load stress. This reduction of dead load shall not be made in proportioning members subject to alternate stresses.

113.—Combined Stresses.—Members subject to both axial and bending stresses shall be proportioned so that the combined fiber stresses will not exceed the unit stress for tension given in Article 201. In members continuous over panel points, three-fourths of the bending stress computed as for simple beams shall be added to the axial stress.

114.—For stresses produced by combination of longitudinal or lateral and wind forces with live load, dead load, impact, and centrifugal force, the unit stresses may be increased 25% over those specified in Article 201. When secondary stresses are included, the unit stress may be increased 33⅓ per cent. In no case shall the section be less than that required for dead load, live load, impact, and centrifugal force at the unit stresses specified in Article 201, or less than that required if secondary stresses are not considered.

115.—Secondary Stresses.—Secondary stresses shall be avoided where possible in designing and detailing. In ordinary trusses without sub-paneling, secondary stresses due to distortion need not be considered in any member the width of which measured parallel with the plane of flexure is less than one-tenth of its length. All other secondary stresses shall be computed.

Section 2.—Unit Stresses

201.—The unit stresses to be used in proportioning the several parts of the structure shall be as follows:

Allowable Stresses for Structural and Rivet Steel:

	Kips per* square inch.
Tension	16.0
Compression (one diameter)	16.0
Compression on columns: $p = \dfrac{16.0}{1 + \dfrac{l^2}{13\,500\,r^2}}$ in which: p = allowable unit stress; l = length of member, in inches; r = least radius of gyration of member, in inches; but not to exceed	14.0
Bending in extreme fibers of rolled shapes, built sections, and girders, net section	16.0
Bending in extreme fibers of pins	24.0
Shear in plate-girder and I-beam webs, net sections	12.0
Shear in pins and power-driven rivets	12.0
Shear in turned bolts and hand-driven rivets	10.0
Bearing on pins, power-driven rivets, outstanding legs of stiffener angles, and other steel parts in contact	24.0
Bearing on turned bolts and hand-driven rivets	20.0
Bearing on countersunk rivets. Only one-half the countersink shall be computed as bearing surface.	
Bearing on rollers per linear inch	0.6 d

* 1 k. = 1 000 lb.

in which,

d = diameter of roller, in inches.

For cast-steel shoes and pedestals, the allowable unit stress for structural steel will apply.

For members composed of steel of greater strength than structural grade the allowable stresses may be increased in proportion to the higher yield point of the stronger steel, provided the yield point is not more than 70% of the ultimate strength. In the column formula, the fractional portion of the denominator should be increased in the same ratio.

202.—Limiting Length of Members.—The lengths of main compression members shall not exceed 100 times their least radius of gyration, and those for wind and sway-bracing 120 times their least radius of gyration.

The lengths of riveted tension members shall not exceed 200 times their least radius of gyration.

203.—Allowable Fiber Stress on Wooden Cross-Ties.—(The maximum wheel load with 100% impact, will be distributed over three ties):

	Kips per square inch.
White oak and dense yellow pine	2.0
Dense Douglas fir	1.5
White pine, ordinary yellow pine, and spruce	1.2

204.—Allowable Pressure on Masonry.—

	Kips per square inch.
Granite masonry	0.8
Limestone and sandstone (good quality)	0.4
Concrete (1:2:4)	0.6

Section 3.—Details of Design

301.—Thickness of Material.—Material less than $\frac{3}{8}$ in. in thickness shall not be used, except for fillers.

302.—Open Details.—Details shall be arranged so as to give free access for inspection and painting. Water-pockets shall be avoided.

303.—Rigid Members.—Hip verticals and members performing similar functions, and the two end panels of the bottom chords of single-track pin-connected bridges preferably shall be rigid.

304.—Viaduct Struts.—The struts at the base of viaduct towers shall be of sufficient strength to slide the movable shoes when the track is unloaded.

305.—Net Sections.—Net sections shall be used in all cases in calculating tension members; and, in deducting rivet holes, they shall be taken as $\frac{1}{8}$ in. larger than the nominal diameter of the rivet. The weakening effect of a staggered rivet shall be allowed for by deducting from the transverse section a strip, w in width, as given by the formula:

$$w = h - \frac{s^2}{4\,g},$$

in which,

w = width, in inches, of strip to be deducted;
h = diameter of rivet hole, in inches;
s = stagger, or longitudinal spacing of rivet with respect to rivet on last gauge line, in inches;
g = distance between gauge lines, or transverse spacing, in inches.

306.—Pin-connected riveted tension members shall have a net section back of the pin-hole, parallel to the axis of the member, of not less than the required net section of the body of the member, and shall have a net section through the

pin-hole at least one-third larger than the required section. Riveted tension members shall be stitch-riveted where necessary to make a compact member.

307.—Maximum Length of Rivet.—The total thickness of plates connected by rivets shall not exceed six times the nominal diameter of the rivet to be used.

308.—Pitch of Rivets.—Rivets shall be proportioned by their nominal diameter. They shall not be spaced, center to center, closer than three diameters nor farther apart, in the direction of the stress, than sixteen times the thickness of the thinnest plate connected, nor farther apart, at right angles to the line of stress, than thirty times that thickness, except in the cover-plates of compression members, where the spacing may be forty times the thickness of the thinnest plate. The pitch of rivets at ends of built compression members shall not be more than four diameters, for a distance of $1\frac{1}{2}$ times the width of the member.

309.—Edge Distance of Rivets.—Rivets shall not be spaced closer to sheared edges than $1\frac{1}{4}$ diameters, nor to rolled or planed edges, than $1\frac{1}{2}$ diameters, nor farther from the edge than eight times the thickness of the plate.

310.—Built Chord Splices.—Built chords subjected to compression only, when faced for bearings, shall be spliced on four sides sufficient to hold the abutting members accurately in place and to transmit at least 25% of the stress through splice plates. Generally, they shall be spliced as near to panel points as practicable. All other joints shall be fully spliced.

311.—Extra Rivets in Packing.—Where stress is transmitted through filler-plates, they shall be extended for extra riveting.

312.—Latticing Compression Members.—All segments of members in compression, connected by latticing only, shall have batten-plates at each end. The thickness of such plates shall not be less than one-fortieth of the distance between the rivets connecting them to the compression member. In no case shall the length of such batten-plates be less than $1\frac{1}{4}$ times the width of the member. Where intermediate batten-plates are used, their length shall be at least three-quarters of the width of the member.

313.—The distance between the connections of latticing shall be such that the individual members between them, composing the column shall be relatively stronger than the column as a whole.

314.—Single Lattice-Bars.—Single lattice-bars shall generally be inclined at an angle of 60° with the axis of the member, and double lattice-bars at an angle of 45°, riveted at the intersection. Single lattice-bars shall have a thickness of not less than one-fortieth, and double lattice-bars not less than one-sixtieth, of the distance between rivets connecting them to the compression member.

315.—The latticing of compression members shall be proportioned to resist shearing stress normal to the member not less than that calculated by the formula:

$$R = \frac{P\,l}{4\,000\,y}$$

in which,

R = normal shearing stress, in pounds;
P = strength of column as a compression member, expressed in pounds;
l = length of column, in inches;
y = distance from neutral axis to extreme fiber, in inches.

In a compression member with a cover-plate, the cover-plate will be assumed to take one-half the shear.

316.—The diameter of the rivets shall not exceed one-third the width of the bar.

317.—Reinforcing Pin-Holes.—Where necessary, pin-holes, shall be reinforced by plates. These plates shall be connected so as to distribute the bearing pressure proportionately over the full cross-section with a minimum of eccen-

tricity. At least one full-width plate on each segment shall extend at least 6 in. beyond the near edge of the batten-plate.

318.—Forked Ends.—Forked ends of compression members should be avoided. Where forked ends are used, a sufficient number of pin-plates shall be provided to give each jaw the full strength of the compression member. At least one of these plates shall extend to the far edge of the batten-plates, and the others not less than 6 in. beyond the near edge of the batten-plates.

319.—Floor-Beam Connections.—Floor-beams preferably shall be square to the girders or trusses, and shall be riveted directly to the girders or to the posts of the trusses.

320.—Lateral Bracing.—Through bridges shall be provided with post brackets at the intermediate panel points, of sufficient strength to maintain the panel in a vertical position under the specified wind pressure. When the height of the top chord is more than 26 ft. above the floor, an overhead system of sway-bracing shall be used.

321.—In either deck or through bridges, the end sway-bracing shall be proportioned to carry the entire upper lateral stress to the support, through the end posts of the truss.

322.—Rigid lateral bracing is preferred and no connection shall have less than three rivets.

323.—Plate Girders.—Plate girders shall be proportioned by assuming that the flanges are concentrated at their centers of gravity, but in no case beyond the back of the flange angles. One-eighth of the gross section of the web may be considered as flange area, provided the web is properly spliced to transmit its bending moment. For unusual sections, the net section modulus shall be used.

324.—Compression Flanges.—The gross section of compression flanges of plate girders or I-beams shall not be less than the gross section of the tension flanges, but the stress per square inch shall not exceed:

$$16\,000 - 150\,\frac{l}{b},$$

in which,

l = length of unsupported flange, between lateral connections or knee-braces, in inches; and

b = width of flange in inches.

325.—When flange plates are used, at least one plate on each flange shall extend the full length of the girder, and on through bridges, an end and corner cover-plate shall be used. Any additional flange plates shall be of such length as to allow two rows of rivets of the regular pitch to be placed at each end of the plate, beyond the theoretical point required, and there shall be a sufficient number of rivets at the ends of each plate to transmit its stress value before the theoretical point of the next outside plate is reached.

326.—Flange plates which are spliced, shall be properly covered by extra material equal in section to the material spliced. Flange angles shall have angle splices. There shall be a sufficient number of rivets on each side of the splice to transmit the stress value of the parts cut.

327.—The webs of plate girders, wherever cut, shall be fully spliced for shear and bending.

328.—Stiffeners.—Stiffener angles shall be placed at end bearings and at points of concentrated load. Such stiffeners shall not be crimped and shall have outstanding legs proportioned for bearing and extending as nearly as practicable to the edge of the flange angles.

329.—Intermediate Stiffeners.—Webs shall be stiffened by angles riveted thereto in pairs, with outstanding legs not exceeding sixteen times their thickness and not less than 2 in. plus one-thirtieth of the depth of the girder. Inter-

mediate stiffeners shall be placed at intervals not exceeding the depth of the web, or 6 ft.

330.—If the depth of the web between the flange angles or side-plates is less than fifty times the thickness of the web, intermediate stiffeners may be omitted.

331.—Expansion Rollers.—All bridges, 80 ft. or more in length, that bear on masonry, shall be provided with pin-bearing bolsters, and, at one end, shall have turned rollers not less than 4 in. in diameter, placed between two planed surfaces. Spans of less than 80 ft. shall be free to move at one end on planed surfaces without rollers.

332.—The nests of rollers shall be designed so as to prevent displacement.

333.—Bearings Anchored.—Trusses shall be secured against side motion on bearing plates and rollers. The bolsters shall be joined to the trusses and the bearing plates secured to the underlying supports by bolts. Anchor-bolts shall not be less than 1¼ in. in diameter, and shall extend not less than 18 in. into the masonry. Washers shall be used under the nut. Anchor-bolts subjected to tension, as in viaduct towers, shall engage 50% more masonry than is sufficient to counteract the uplift.

334.—Eye-Bar Packing.—Eye-bars shall not be packed out of line with the axis of the member more than $\frac{1}{16}$ in. to 1 ft. Members packed on pins shall be held against side movement.

335.—Camber.—The length of truss members shall be such that the camber will be equal to the deflection produced by the dead load plus the full train load, without impact.

Section 4.—Workmanship

401.—General.—All workmanship shall be equal to the best practice in modern bridge shops. The methods used and the character of the work done shall be such that the strength of the various members, as designed, shall be fully realized.

402.—Straightening.—When necessary, the material shall be straightened by methods that will avoid any excessive local internal stresses. Sharp kinks and bends may be cause for rejection.

403.—Shearing.—Sheared plates, exceeding $\frac{3}{4}$ in. in thickness, shall have $\frac{1}{4}$ in. planed from their edges.

404.—Rivet Holes in Structural Steel.—Material forming parts of a member composed of not more than five thicknesses of structural steel may be punched $\frac{1}{16}$ in. larger than the nominal size of the rivets, whenever the thickness of the metal is equal to, or less than, the size of the rivets, plus $\frac{1}{8}$ in. When the metal is thicker than the size of the rivet, plus $\frac{1}{8}$ in., the holes shall be drilled.

405.—Material forming parts of a member composed of more than five thicknesses of structural steel shall be punched, or drilled, with a tool $\frac{3}{16}$ in. smaller than the nominal size of the rivets and reamed to full size after the parts are assembled. When the holes are made after the parts are assembled, they shall be drilled full size.

406.—Rivet Holes in Alloy Steel.—Material forming parts of a member composed of not more than five thicknesses of alloy steel may be punched $\frac{1}{16}$ in. larger than the nominal size of the rivets, whenever the thickness of the metal does not exceed $\frac{1}{2}$ in. When the metal is thicker than $\frac{1}{2}$ in., the holes shall be drilled.

407.—Material forming parts of a member composed of more than five thicknesses of alloy steel shall be punched, or drilled, with a tool $\frac{3}{16}$ in. smaller than the size of the rivets and reamed to full size after the parts are assembled. When the holes are made after the parts are assembled, they shall be drilled full size.

408.—Drifting.—Drifting to enlarge unfair holes will not be allowed.

409.—Field Rivet Holes.—All holes for field connections, except those for bracing and similar parts, shall be reamed or drilled to a metal templet, or while the members are temporarily assembled. When the reaming or drilling is done with the pieces assembled, the parts shall be match-marked and a diagram of the marks made.

410.—Dies.—Punch dies shall not be more than $\frac{3}{32}$ in. larger than the punch.

411.—Web-Plates.—Web-plates of girders which have no cover-plates, shall be flush with the backs of the top flange angles or project above them not more than $\frac{1}{8}$ in.

Web-plates of girders that have cover-plates may be $\frac{1}{2}$ in. less in width than the distance back to back of flange angles. The ends of web-plates at splices shall not be more than $\frac{1}{4}$ in. apart.

412.—Splice Plates and Fillers.—Web splices, plates, and fillers shall fit within $\frac{1}{8}$ in. of the flange angles.

413.—Web Stiffeners.—Web stiffeners at bearings and points of concentrated loads on the flanges, shall be faced and brought to bearing against the flange angles.

414.—Lattice-Bars.—The ends of the lattice-bars with single rivets shall be neatly rounded.

415.—Floor-Beams and Stringers.—The main sections of floor-beams and stringers shall be of exact length after riveting, and the end-connection angles shall be accurately set to length, and square.

416.—Turned Bolts.—Generally, the use of bolts instead of rivets shall not be permitted. Wherever bolts are used, the holes shall be reamed parallel and the bolts turned to a driving fit, and the threads shall be entirely outside the holes. A washer not less than $\frac{1}{4}$ in. thick shall be used under the nut and the thread burred.

417.—Abutting Joints.—Abutting joints shall be fitted close. The abutting surfaces of compression joints which depend on contact bearing shall be accurately faced.

418.—Eye-Bars.—Eye-bars shall be made by a process that will produce a uniform product. Welds will not be allowed. The form of the heads may be determined by the dies at the works where the eye-bars are made, if satisfactory to the Engineer, but the manufacturers shall guarantee the bars to break in the body when tested to rupture. The thickness of the head shall not be more than $\frac{1}{8}$ in. greater than that of the bar, except by mutual agreement in special cases.

419.—Pin Clearance.—The diameters of pin-holes shall not exceed those of the pins by more than $\frac{1}{50}$ in. for pins, 8 in. or less in diameter, nor more than $\frac{1}{32}$ in. for larger pins.

420.—Pins and Rollers.—Pins more than 7 in. in diameter shall be forged and annealed. Pins 9 in. or more in diameter shall have a 2-in. longitudinal hole bored through the axis. All pins shall be turned.

421.—Steel Castings.—Steel castings shall be free from large and injurious blow-holes and shall be annealed.

422.—Bed-Plates.—Expansion bed-plates shall be planed true and smooth. The finishing cut of the planer tool shall be in the direction of expansion. Cast wall-plates shall be planed on top.

423.—Screw-Threads.—Screw-threads shall be U. S. Standard. The nuts shall fit tight.

424.—Welds.—Welds will be allowed only where the unit stress is not more than one-half the ordinary specified working stress.

425.—Pilot Nuts.—Two pilot and two driving nuts shall be furnished for each size of pin.

426.—Field Rivets.—Field rivets shall be furnished in excess of the nominal number required to the amount of 10% plus 10 rivets of each size and length.

427.—Shop Painting.—All surfaces that come in contact or that are closed shall receive one coat of approved paint before being assembled. All steel shall be scraped free of scale and shall receive one coat of approved paint before leaving the shops.

428.—All machined surfaces shall receive a coat of white lead and tallow at the shop.

429.—Facilities for Inspection.—The Inspector shall be allowed access to all necessary parts of the works.

430.—Inspector's Authority.—The Inspector shall have the power to reject materials and workmanship which do not fulfill the requirements of these specifications; but, in case of dispute, the Contractor may appeal to the Engineer, whose decision shall be final.

431.—Rejection.—The acceptance of any material or finished members by the Inspector, shall not be a bar to their subsequent rejection if they are found to be defective.

Rejected material and workmanship shall be replaced promptly or made good by the Contractor.

Section 5.—Full-Size Eye-Bar Tests

501.—Material Tests.—When the acceptance of eye-bars depends on the results of full-size tests, the quality of the material, as proved by the specimen tests, shall be determined by the manufacturer.

502.—Number of Full-Size Tests.—The number and size of the bars to be tested shall be specified by the Engineer before the mill order is placed.

503.—Selection of Eye-Bars.—The test bars shall be the same section as the bars that are to be used in the structure and of the same length, if within the capacity of the testing machine. Test bars representing bars too long for the testing machine shall be selected from the full length bar material after the heads on one end have been formed, and shall then be cut and the second head formed to the greatest length that can be tested.

504.—Requirements for Full-Size Tests.—The requirement for full-size eye-bar tests shall be as follows:

	Structural steel annealed eye-bars.
Yield point, in pounds per square inch, minimum.....	30 000
Ultimate strength, in pounds per square inch, minimum.	55 000
Elongation percentage in 18 ft., minimum.....	12

The elongation shall be measured in the body of the bar including the fracture.

505.—Re-Tests.—When a bar fails to meet the requirements, two additional bars of the same size and from the same mill heat may be tested. If two of three bars tested fail, the bars of that size and mill heat shall be rejected.

506.—Payment for Eye-Bars.—Eye-bars thus tested which meet the requirements of these specifications, shall be paid for by the purchaser at the same unit price as for the structure. Bars which fail to meet the requirements of the specifications, and all bars rejected as a result of the tests, shall be at the expense of the Contractor.

Section 6.—Weighing and Shipping

601.—Weight Paid for.—Payment on pound-price contracts shall be based on scale weight, including field rivets. Not more than 1½% of the total computed weight of the structure will be allowed for excess weight.

602.—Any member weighing 2½% less than the computed weight may be rejected.

603.—Marking and Shipping.—Pins and small parts shall be protected for shipment. Rivets and bolts of one length and diameter, also loose nuts or washers of each size, shall be packed separately. The contents of each package shall be marked thereon.

604.—Members weighing more than 5 tons shall have the weight marked thereon.

Section 7.—Structural Steel for Bridges

These specifications conform to the Standard Specifications for Structural Steel for Bridges of the American Society for Testing Materials.

701.—Steel Castings.—The Standard Specifications for Steel Castings (Serial Designation, A27), adopted by the American Society for Testing Materials, shall govern the purchase of steel castings for bridges. Unless otherwise specified, Class B castings, medium grade, shall be used.

I.—Manufacture

702.—Process.—The steel shall be made by the open-hearth process.

II.—Chemical Properties and Tests

703.—Chemical Composition.—The steel shall conform to the following requirements as to chemical composition:

		Structural steel.	Rivet steel.
Phosphorus	Acid	not over 0.06%	not over 0.04 %
	Basic	" " 0.04%	" " 0.04 %
Sulphur		" " 0.05%	" " 0.045%

704.—Ladle Analyses.—An analysis of each melt of steel shall be made by the manufacturer to determine the percentages of carbon, manganese, phosphorus, and sulphur. This analysis shall be made from a test ingot taken during the pouring of the melt. The chemical composition thus determined shall be reported to the purchaser or his representative, and shall conform to the requirements specified in Article 703.

705.—Check Analyses.—Analyses may be made by the purchaser from finished material representing each melt. The phosphorus and sulphur content thus determined shall not exceed that specified in Article 703 by more than 25 per cent.

III.—Physical Properties and Tests

706.—Tension Tests.—

(*a*).—The material shall conform to the requirements as to tensile properties given in Table 1.

TABLE 1.

Properties considered.	Structural steel.	Rivet steel.
Tensile strength, in pounds per square inch	55 000—65 000*	46 000—56 000
Yield point, minimum, per square inch	0.5 tensile strength	0.5 tensile strength
Elongation in 8 in., minimum percentage	$\frac{1\,500\,000\dagger}{\text{Tensile strength}}$	$\frac{1\,500\,000}{\text{Tensile strength}}$
Elongation in 2 in., " "	22	

* See Article 706, Paragraph (*b*).

† See Article 707.

(*b*).—In order to meet the required minimum tensile strength of full-size annealed eye-bars, the purchaser may determine the tensile strength to be obtained in specimen tests; the range shall not exceed 4 000 lb. per sq. in., and the maximum shall not exceed 74 000 lb. per sq. in. The material shall conform to the requirements as to physical properties other than that of tensile strength, specified in Articles 706(*b*), 707(*b*), and 708(*b*).

(*c*).—The yield point shall be determined by the drop of the beam of the testing machine.

707.—Modifications in Elongation.—

(*a*).—For structural steel more than $\frac{3}{4}$ in. in thickness, a deduction from the percentage of elongation in the 8 in. specified in Article 706(*a*), of 0.25% shall be made for each increase of $\frac{1}{32}$ in. of the specified thickness above $\frac{3}{4}$ in., to a minimum of 18 per cent.

(*b*).—For structural steel less than $\frac{5}{16}$ in. in thickness, a deduction from the percentage of elongation in the 8 in. specified in Article 706(*a*), of 1.25% shall be made for each decrease of $\frac{1}{32}$ in. in thickness below $\frac{5}{16}$ in.

708.—Bend Tests.—

(*a*).—The test specimen for plates, shapes, and bars, except as specified in Paragraphs (*b*), (*c*), and (*d*), shall bend cold through 180° without cracking on the outside of the bent portion, as follows: For material $\frac{3}{4}$ in. or less in thickness, flat on itself; for material more than $\frac{3}{4}$ in. to and including $1\frac{1}{4}$ in. in thickness, around a pin the diameter of which is equal to the thickness of the specimen; and for material more than $1\frac{1}{4}$ in. in thickness, around a pin the diameter of which is equal to twice the thickness of the specimen.

(*b*).—The test specimen for eye-bar flats shall bend cold through 180° without cracking on the outside of the bent portion, as follows: For material $\frac{3}{4}$ in. or less in thickness, around a pin the diameter of which is equal to the thickness of the specimen; for material more than $\frac{3}{4}$ in. to and including $1\frac{1}{4}$ in. in thickness, around a pin the diameter of which is equal to twice the thickness of the specimen; and for material more than $1\frac{1}{4}$ in. in thickness, around a pin the diameter of which is equal to three times the thickness of the specimen.

(*c*).—The 1 in. by $\frac{1}{2}$-in. test specimen for pins, rollers, and other bars, when prepared as specified in Article 709, shall bend cold through 180° around a pin 1 in. in diameter, without cracking on the outside of the bent portion.

(*d*).—The test specimen for rivet steel shall bend cold through 180° flat on itself, without cracking on the outside of the bent portion.

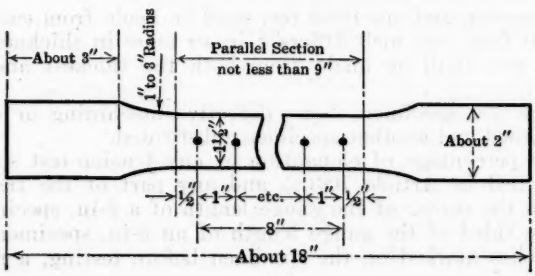


FIG. 4.

709.—Test Specimens.—

(*a*).—Test specimens shall be prepared for testing from the material in its rolled or forged condition, except when it is specified to be annealed, in which case the test specimens shall be prepared from the material as annealed for use, or from a short length of a full section similarly treated.

(*b*).—Test specimens shall be taken longitudinally and, except as specified in Paragraphs (*d*), (*e*), and (*f*), shall be of the full thickness or diameter of the material as rolled.

(*c*).—Test specimens for plates, shapes, and flats may be machined to the form and dimensions shown in Fig. 4, or with both edges parallel; except that bend-test specimens for eye-bar flats may have three rolled sides.

(*d*).—Tension-test specimens for plates and eye-bar flats more than 1½ in. in thickness, and bend-test specimens for plates more than 1½ in. in thickness, may be machined to a thickness or diameter of at least ¾ in. for a length of at least 9 in.

(*e*).—Test specimens for bars more than 1½ in. in thickness or diameter may be machined to a thickness or diameter of at least ¾ in. for a length of at least 9 in.; or tension-test specimens may conform to the dimensions shown in Fig. 5, in which case the ends shall be of a form to fit the holders of the testing machine in such a way that the load shall be axial. Bend-test specimens may be 1 in. by ½ in. in section.

(*f*).—Tension-test specimens for pins and rollers shall conform to the dimensions shown in Fig. 5. In this case, the ends shall be of a form to fit the holders of the testing machine in such a way that the load shall be axial. Bend-test specimens shall be 1 in. by ½ in. in section.

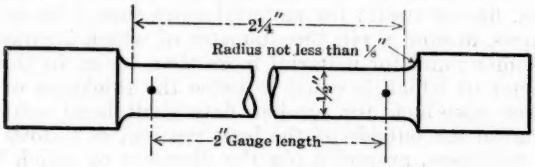


FIG. 5.

(*g*).—The tension-test specimen shown in Fig. 5 and the 1 in. by ½-in. bend-test specimen for pins and rollers shall be taken so that the axis is 1 in. from the surface; and for other bars more than 1½ in. in thickness or diameter, midway between the center and surface.*

(*h*).—The machined sides of rectangular bend-test specimens may have the corners rounded to a radius of not more than ⅛ in.

(*i*).—Test specimens for rivet bars which have been cold drawn, shall be normalized before testing.

710.—Number of Tests.—

(*a*).—One tension and one bend test shall be made from each melt; except that if material from one melt differs ⅜ in. or more in thickness, one tension and one bend test shall be made from both the thickest and the thinnest material rolled.

(*b*).—If any test specimen shows defective machining or develops flaws, it may be discarded and another specimen substituted.

(*c*).—If the percentage of elongation of any tension-test specimen is less than that specified in Article 706(*a*) and any part of the fracture is more than ¾ in. from the center of the gauge length of a 2-in. specimen, or is outside the middle third of the gauge length of an 8-in. specimen, as indicated by scribe scratches marked on the specimen before testing, a re-test shall be allowed.

IV.—Permissible Variations in Weight and Thickness

711.—The cross-section or weight of each piece of steel shall not vary more than 2.5% from that specified, except in the case of sheared plates which

* The gauge length, parallel portions, and fillets shall be as shown in Fig. 5, but the ends may be of any form which will fit the holders of the testing machine.

shall be covered by the following permissible variations (1 cu. in. of rolled steel is assumed to weigh 0.2833 lb.):

(*a*).—When ordered to weight per square foot: The weight of each lot* in each shipment shall not vary from the weight ordered more than the amount given in Table 2.

(*b*).—When ordered to thickness: The thickness of each plate shall not vary more than 0.01 in. less than that ordered.

The overweight of each lot† in each shipment shall not exceed the amount given in Table 3.

TABLE 2.—PERMISSIBLE VARIATIONS OF PLATES ORDERED TO WEIGHT.*

Ordered weight, in pounds per square foot.	Permissible Variations in Average Weights per Square Foot of Plates for Widths Given, Expressed in Percentages of Ordered Weights:																	
	Under 48 in.		48 to 60 in., excl.		60 to 72 in., excl.		72 to 84 in., excl.		84 to 96 in., excl.		96 to 108 in., excl.		108 to 120 in., excl.		120 to 132 in., excl.		132 in., or over.	
	Over	Under	Over	Under	Over	Under	Over	Under	Over	Under	Over	Under	Over	Under	Over	Under	Over	Under
Under 5	5	3	5.5	3	6	3	7	3	....	....	....	....	....	....	....	....	....	....
5 to 7.5, excl.	4.5	3	5	3	5.5	3	6	3	....	....	....	....	....	....	....	....	....	....
7.5 " 10, "	4	3	4.5	3	5	3	5.5	3	6	3	7	3	8	3	....	....	....	....
10 " 12.5, "	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3	7	3	8	3	9	3
12.5 " 15, "	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3	7	3	8	3
15 " 17.5, "	2.5	2.5	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3	7	3
17.5 " 20, "	2.5	2	2.5	2.5	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3
20 " 25, "	2	2	2.5	2	2.5	2.5	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3
25 " 30, "	2	2	2	2	2.5	2	2.5	2.5	3	2.5	3.5	3	4	3	4.5	3	5	3
30 " 40, "	2	2	2	2	2	2	2.5	2	2.5	2.5	3	2.5	3.5	3	4	3	4.5	3
40, or over	2	2	2	2	2	2	2	2	2.5	2	2.5	2.5	3	2.5	3.5	3	4	3

* The weight per square foot of individual plates shall not vary from the ordered weight by more than one and one-third times the amount given in Table 2.

TABLE 3.—PERMISSIBLE OVERWEIGHTS OF PLATES ORDERED TO THICKNESS.

Ordered thickness, in inches.	Permissible Excess in Average Weights per Square Foot of Plates for Widths Given, Expressed in Percentages of Nominal Weights:								
	Under 48 in.	48 to 60 in., excl.	60 to 72 in., excl.	72 to 84 in., excl.	84 to 96 in., excl.	96 to 108 in., excl.	108 to 120 in., excl.	120 to 132 in., excl.	132 in. or over.
Under 1/8	9	10	12	14	....	....	....	....	....
1/8 to 3/16 excl.	8	9	10	12	....	....	....	....	....
3/16 " 1/4 "	7	8	9	10	12	....	....	....	....
1/4 " 5/16 "	6	7	8	9	10	12	14	16	19
5/16 " 3/8 "	5	6	7	8	9	10	12	14	17
3/8 " 7/16 "	4.5	5	6	7	8	9	10	12	15
7/16 " 1/2 "	4	4.5	5	6	7	8	9	10	13
1/2 " 5/8 "	3.5	4	4.5	5	6	7	8	9	11
5/8 " 3/4 "	3	3.5	4	4.5	5	6	7	8	9
3/4 " 1 "	2.5	3	3.5	4	4.5	5	6	7	8
1, or over	2.5	2.5	3	3.5	4	4.5	5	6	7

* The term "lot" applied to Table 2 means all the plates of each group width and group weight.

† The term "lot" applied to Table 3 means all the plates of each group width and group thickness.

V.—Finish

712.—The finished material shall be free from injurious defects and shall have a workmanlike finish.

VI.—Marking

713.—The name or brand of the manufacturer and the melt number shall be legibly stamped or rolled on all finished material, except that rivet and lattice-bars and other small sections, when loaded for shipment, shall be properly separated and marked for identification. The identification marks shall be legibly stamped on the end of each pin and roller. The melt number shall be legibly marked, by stamping if practicable, on each test specimen.

VII.—Inspection and Rejection

714.—Inspection.—The Inspector representing the purchaser shall have free entry, at all times while work on the contract of the purchaser is being performed, to all parts of the manufacturer's works which concern the manufacture of the material ordered. The manufacturer shall afford the Inspector, free of cost, all reasonable facilities to satisfy him that the material is being furnished in accordance with these specifications. All tests (except check analyses) and inspection shall be made at the place of manufacture prior to shipment, unless otherwise specified, and shall be conducted so as not to interfere unnecessarily with the operation of the works.

715.—Rejection.—

(*a*).—Unless otherwise specified, any rejection based on tests made in accordance with Article 705 shall be reported within five working days from the receipt of samples.

(*b*).—Material which shows injurious defects subsequent to its acceptance at the manufacturer's works will be rejected, and the manufacturer shall be notified.

716.—Rehearing.—Samples tested in accordance with Article 705, which represent rejected material, shall be preserved for two weeks from the date of the test report. In case of dissatisfaction with the results of the tests, the manufacturer may make claim for a rehearing within that time.

Section 8.—Structural Nickel Steel

These specifications conform to the Standard Specifications for Structural Nickel Steel of the American Society for Testing Materials.

I.—Manufacture

801.—Process.—The steel shall be made by the open-hearth process.

802.—Discard.—A sufficient discard shall be made from each ingot intended for eye-bars to secure freedom from injurious piping and undue segregation.

II.—Chemical Properties and Tests

803.—Chemical Composition.—The steel shall conform to the following requirements as to chemical composition:

		Structural steel.	Rivet steel.
Carbon		not over 0.45%	not over 0.30%
Manganese		" " 0.70%	" " 0.60%
Phosphorus	Acid	" " 0.05%	" " 0.04%
	Basic	" " 0.04%	" " 0.03%
Sulphur		" " 0.05%	" " 0.045%
Nickel		not under 3.25%	not under 3.25%

804.—Ladle Analyses.—An analysis of each melt of steel shall be made by the manufacturer to determine the percentages of the elements specified in Article 803. This analysis shall be made from a test ingot taken during the pouring of the melt. The chemical composition thus determined shall be reported to the purchaser or his representative, and shall conform to the requirements specified in Article 803.

805.—Check Analyses.—Analyses may be made by the purchaser from finished material representing each melt. The chemical composition thus determined shall conform to the requirements specified in Article 803.

III.—Physical Properties and Tests

806.—Tension Tests.—

(*a*).—The material shall conform to the requirements as to tensile properties given in Table 4.

TABLE 4.

Properties considered.	Rivet steel.	Plates, shapes, and bars.	Eye-bar flats and rollers, ‡ unannealed.	Eye-bar flats* and plus, ‡ annealed.
Tensile strength, in pounds per square inch	70 000-80 000	85 000-100 000	95 000-110 000	90 000-105 000
Yield point, minimum, in pounds per square inch	45 000	50 000	55 000	52 000
Elongation in 8 in., minimum, percentage	1 500 000 / Tensile strength	1 500 000† / Tensile strength	1 500 000† / Tensile strength	20
Elongation in 2 in., minimum, percentage	.....	.....	16	20
Reduction of area, minimum, percentage	40	25	25	35

*Tests of annealed specimens of eye-bar flats shall be made for information only.
†See Article 807.
‡Elongation shall be measured in 2 in.

(*b*).—The yield point shall be determined by the drop of the beam of the testing machine.

807.—Modifications in Elongation.—For plates, shapes, and unannealed bars of more than 1 in. in thickness, a deduction from the percentage of elongation specified in Article 806(*a*), of 0.25% shall be made for each increase of $\frac{1}{32}$ in. of the specified thickness above 1 in., to a minimum of 14 per cent.

808.—Character of Fracture.—All broken tension-test specimens shall show either a silky or a very fine granular fracture, of uniform color, and free from coarse crystals.

809.—Bend Tests.—

(*a*).—The test specimen for plates, shapes, and bars shall bend cold through 180° without cracking on the outside of the bent portion, as follows: For material $\frac{3}{4}$ in. or less in thickness, around a pin the diameter of which is equal to the thickness of the specimen; and for material more than $\frac{3}{4}$ in. in thickness, around a pin the diameter of which is equal to twice the thickness of the specimen.

(*b*).—The test specimen for pins and rollers shall bend cold through 180° around a pin 1 in. in diameter, without cracking on the outside of the bent portion.

(*c*).—The test specimen for rivet steel shall bend cold through 180°, flat on itself, without cracking on the outside of the bent portion.

810.—Drift Tests.—Punched rivet holes pitched two diameters from a planed edge shall stand drifting until the diameter is enlarged 50%, without cracking the metal.

811.—Test Specimens.—

(*a*).—Test specimens shall be prepared for testing from the material in its rolled or forged condition, except when it is specified to be annealed, in which case, the test specimens shall be prepared from the material as annealed for use, or from a short length of a full section similarly treated.

(*b*).—Test specimens shall be taken longitudinally and, except as specified in Paragraphs (*d*), (*e*), and (*f*), shall be of the full thickness or diameter of material as rolled.

(*c*).—Test specimens for plates, shapes, and flats may be machined to the form and dimensions shown in Fig. 4, or with both edges parallel; except that bend-test specimens for eye-bar flats may have three-rolled sides.

(*d*).—Tension-test specimens for plates and eye-bar flats more than 1½ in. in thickness, and bend-test specimens for plates more than 1½ in. in thickness, may be machined to a thickness or diameter of at least ¾ in. for a length of at least 9 in.

(*e*).—Test specimens for bars more than 1½ in. in thickness or diameter may be machined to a thickness or diameter of at least ¾ in. for a length of at least 9 in.; or tension-test specimens may conform to the dimensions shown in Fig. 5, in which case, the ends shall be of a form to fit the holders of the testing machine in such a way that the load shall be axial. Bend-test specimens may be 1 in. by ½ in. in section.

(*f*).—Tension-test specimens for pins and rollers shall conform to the dimensions shown in Fig. 5. In this case, the ends shall be of a form to fit the holders of the testing machine in such a way that the load shall be axial. Bend-test specimens shall be 1 in. by ½ in. in section.

(*g*).—The tension-test specimen shown in Fig. 5 and the 1 in. by ½-in. bend-test specimen for pins and rollers shall be taken so that the axis is 1 in. from the surface; and for other bars more than 1½ in. in thickness or diameter, midway between the center and surface.

(*h*).—The machined sides of rectangular bend-test specimens may have the corners rounded to a radius of not more than ⅛ in.

(*i*).—Test specimens for rivet bars which have been cold drawn, shall be normalized before testing.

812.—Number of Tests.—

(*a*).—One tension and one bend test shall be made from each melt; except that if material from one melt differs ⅜ in. or more in thickness, one tension and one bend test shall be made from both the thickest and the thinnest material rolled.

(*b*).—If any test specimen shows defective machining or develops flaws, it may be discarded and another specimen substituted.

(*c*).—If the percentage of elongation of any tension-test specimen is less than that specified in Article 806(*a*) and any part of the fracture is more than ¾ in. from the center of the gauge length of a 2-in. specimen, or is outside the middle third of the gauge length of an 8-in. specimen, as indicated by scribe scratches marked on the specimen before testing, a re-test shall be allowed.

IV.—Permissible Variations in Weight and Thickness

813.—The cross-section or weight of each piece of steel shall not vary more than 2.5% from that specified; except in the case of sheared plates, which shall be covered by the following permissible variations (1 cu. in. of rolled steel is assumed to weigh 0.2833 lb.):

(*a*).—When ordered to weight per square foot: The weight of each lot* in each shipment shall not vary from the weight ordered more than the amount given in Table 2.

* The term "lot" applied to Table 2 means all the plates of each group width and group weight.

(*b*).—When ordered to thickness: The thickness of each plate shall not vary more than 0.01 in. under that ordered.

The overweight of each lot* in each shipment shall not exceed the amount given in Table 3.

V.—Finish

814.—The finished material shall be free from injurious defects and shall have a workmanlike finish.

VI.—Marking

815.—The name or brand of the manufacturer and the melt number shall be legibly stamped or rolled on all finished material, except that rivet and lattice-bars and other small sections, when loaded for shipment, shall be properly separated and marked for identification. The identification marks shall be legibly stamped on the end of each pin and roller. The melt number shall be legibly marked, by stamping if practicable, on each test specimen.

VII.—Inspection and Rejection

816.—Inspection.—The Inspector representing the purchaser shall have free entry, at all times while work on the contract of the purchaser is being performed, to all parts of the manufacturer's works which concern the manufacture of the material ordered. The manufacturer shall afford the Inspector, free of cost, all reasonable facilities to satisfy him that the material is being furnished in accordance with these specifications. All tests (except check analyses) and inspection shall be made at the place of manufacture prior to shipment, unless otherwise specified, and shall be conducted so as not to interfere unnecessarily with the operation of the works.

817.—Rejection.—

(*a*).—Unless otherwise specified, any rejection based on tests made in accordance with Article 805 shall be reported within five working days from the receipt of samples.

(*b*).—Material which shows injurious defects subsequent to its acceptance at the manufacturer's works will be rejected, and the manufacturer shall be notified.

818.—Rehearing.—Samples tested in accordance with Article 805, which represent rejected material, shall be preserved for two weeks from the date of the test report. In case of dissatisfaction with the results of the tests, the manufacturer may make claim for a rehearing within that time.

VIII.—Full-Size Tests

819.—Tests of Eye-bars.—

(*a*).—Full-size tests of annealed eye-bars shall conform to the following requirements as to tensile properties:

Tensile strength, in pounds per square inch...	85 000—100 000
Yield point, minimum, in pounds per square inch	48 000
Elongation in 18 ft., minimum, percentage....	10
Reduction of area, " "	30

(*b*).—The yield point shall be determined by the halt of the gauge of the testing machine.

* The term "lot" applied to Table 3 means all the plates of each group width and group thickness.

APPENDIX

Tables 5 to 20, inclusive, and Fig. 6, are submitted in this Appendix, in order to facilitate the use of the Specifications. Because of the fact that two engine loadings are presented, it was thought desirable to furnish working tables for both loadings and a table of coefficients for use in converting E-rating to equivalent M-rating, and *vice versa*.

CONTENTS

TABLE		PAGE
5.	Shearing and Bearing Values of Power-Driven Rivets	78
6.	Shearing and Bearing Values of Hand-Driven Rivets and Turned Bolts	78
7.	Bending Moments and Bearing Values for Pins	79
8.	Working Stresses for Columns	80
9.	Coefficients for Columns	80
10.	Moments for Class E-10 Engine Loading	81
11.	Wheel Determining Maximum Moment for Class E Engine Loading	82
12.	Moments, Shears, and Reactions for Class E-10 Engine Loading	83
13.	Maximum Moments for Class E-10 Engine Loading	85
14.	Maximum Shears for Class E-10 Engine Loading	91
15.	Conversion Table, Class E to Class M Engine Loading	96
16.	Moments for Class M-10 Engine Loading	97
17.	Wheel Determining Maximum Moment for Class M Engine Loading	98
18.	Moments, Shears, and Reactions for Class M-10 Engine Loading	99
19.	Maximum Moments for Class M-10 Engine Loading	101
20.	Maximum Shears for Class M-10 Engine Loading	107

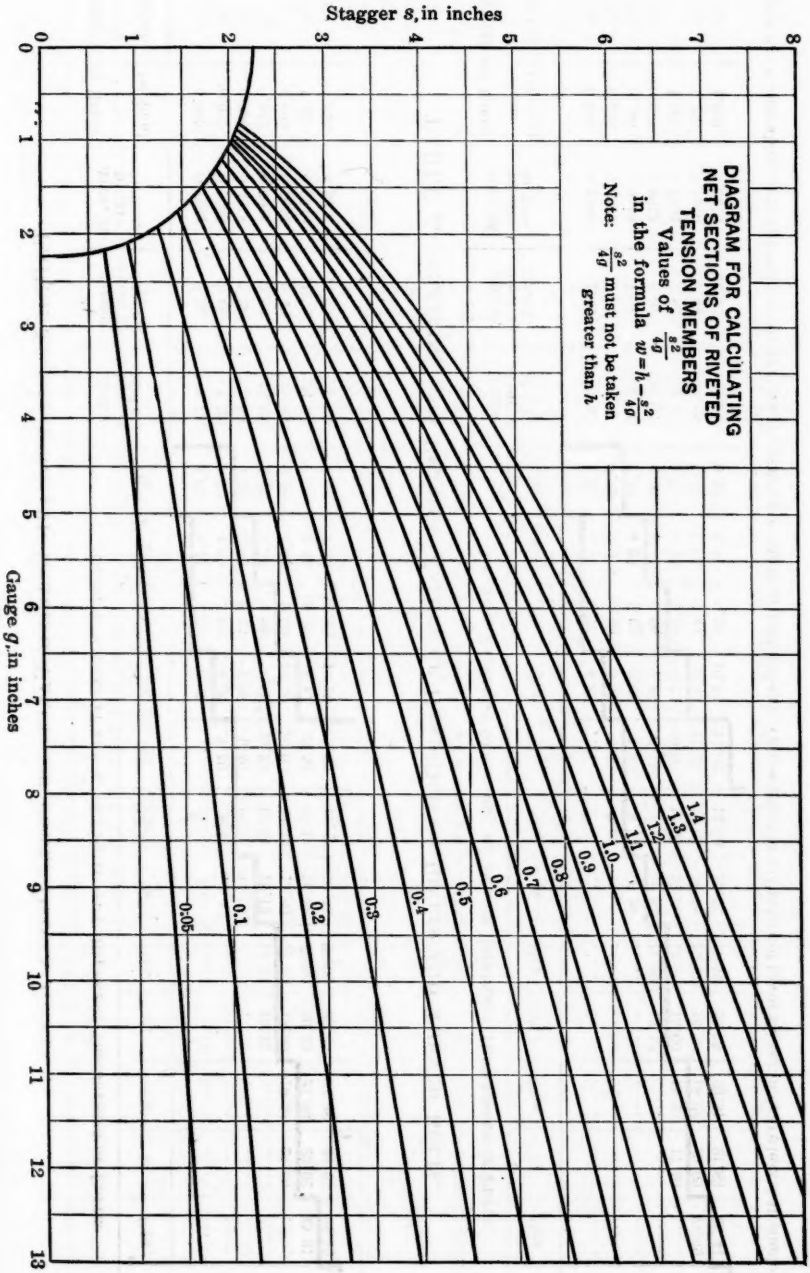


DIAGRAM FOR CALCULATING NET SECTIONS OF RIVETED TENSION MEMBERS
Values of $\frac{s^2}{4g}$ in the formula $w = h - \frac{s^2}{4g}$
Note: $\frac{s^2}{4g}$ must not be taken greater than h.
Stagger s, in inches
Gauge g, in inches
1.4
1.3
1.2
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05

Fig. 6.

TABLE 5.—Shearing and Bearing Values of Power-Driven Rivets.

Diameter of Rivet.		Area, in square inches.	Single shear @ 12.0 k.	Double shear @ 12.0 k.	Bearing Values at 24.0 k. per sq. in. for Different Thicknesses of Plate.												
Fraction.	Decimal.				¼	5/16	3/8	7/16	½	9/16	5/8	11/16	¾	13/16	⅞	15/16	1
½	0.500	0.1964	2.36	4.71	3.00	3.75	4.50	5.25	....	.....	.....	.....	.....	.....	.....	.....	.....
5/8	0.625	0.3068	3.68	7.36	3.75	4.69	5.63	6.56	7.50	8.44	.....	.....	.....	.....	.....	.....	.....
¾	0.750	0.4418	5.30	10.60	4.50	5.63	6.75	7.88	9.00	10.13	11.25	.....	.....	.....	.....	.....	.....
⅞	0.875	0.6013	7.22	14.43	5.25	6.56	7.88	9.19	10.50	11.81	13.13	14.44	15.75	.....	.....	.....	.....
1	1.000	0.7854	9.43	18.85	6.00	7.50	9.00	10.50	12.00	13.50	15.00	16.50	18.00	19.50	21.00	.....	.....
1⅛	1.125	0.9940	11.93	23.96	6.75	8.44	10.12	11.81	13.50	15.19	16.88	18.56	20.25	21.94	23.62	25.31	27.00
1¼	1.250	1.2272	14.73	28.45	7.50	9.38	11.25	13.12	15.00	16.88	18.75	20.62	22.50	24.38	26.25	28.12	30.00

TABLE 6.—Shearing and Bearing Values of Hand-Driven Rivets and Turned Bolts.

Diameter of Rivet.		Area, in square inches.	Single shear @ 10.0 k.	Double shear @ 10.0 k.	Bearing Values at 20.0 k. per sq. in. for Different Thicknesses of Plate.												
Fraction.	Decimal.				¼	5/16	3/8	7/16	½	9/16	5/8	11/16	¾	13/16	⅞	15/16	1
½	0.500	0.1964	1.96	3.93	2.50	3.13	3.75	4.38	....	.....	.....	.....	.....	.....	.....	.....	.....
5/8	0.625	0.3068	3.07	6.14	3.13	3.91	4.69	5.47	6.25	7.03	.....	.....	.....	.....	.....	.....	.....
¾	0.750	0.4418	4.42	8.84	3.75	4.69	5.63	6.56	7.50	8.44	9.38	.....	.....	.....	.....	.....	.....
⅞	0.875	0.6013	6.01	12.03	4.38	5.47	6.56	7.66	8.75	9.84	10.94	12.03	13.13	.....	.....	.....	.....
1	1.000	0.7854	7.85	15.71	5.00	6.25	7.50	8.75	10.00	11.25	12.50	13.75	15.00	16.25	17.50	.....	.....
1⅛	1.125	0.9940	9.94	19.88	5.63	7.03	8.44	9.85	11.25	12.65	14.06	15.47	16.88	18.28	19.69	21.09	.....
1¼	1.250	1.2272	12.27	24.54	6.25	7.81	9.38	10.94	12.50	14.06	15.62	17.19	18.75	20.31	21.88	23.44	25.00

Note.—Values above or to the right of upper zigzag lines are greater than double shear. Values below or to the left of lower zigzag lines are less than single shear.

TABLE 7.—BENDING MOMENTS AND BEARING VALUES OF PINS AT 24.0 K. PER SQ. IN.

Diameter, in inches.	Area, in square inches.	Maximum moment, in inch-kips.	Bearing value, per linear inch, in kips.	Diameter, in inches.	Area, in square inches.	Maximum moment, in inch-kips.	Bearing value, per linear inch, in kips.
1	0.785	2.4	24.0	7	38.485	808.2	168.0
1¼	1.227	4.6	30.0	7¼	41.282	897.9	174.0
1½	1.767	8.0	36.0	7½	44.179	994.0	180.0
1¾	2.405	12.6	42.0	7¾	47.173	1 096.8	186.0
2	3.142	18.8	48.0	8	50.265	1 206.4	192.0
2¼	3.976	26.8	54.0	8¼	53.456	1 323.0	198.0
2½	4.909	36.8	60.0	8½	56.745	1 447.0	204.0
2¾	5.940	49.0	66.0	8¾	60.132	1 578.5	210.0
3	7.069	63.6	72.0	9	63.617	1 717.7	216.0
3¼	8.296	80.9	78.0	9¼	67.201	1 864.8	222.0
3½	9.621	101.0	84.0	9½	70.882	2 020.1	228.0
3¾	11.045	124.3	90.0	9¾	74.662	2 183.9	234.0
4	12.566	150.8	96.0	10	78.540	2 356.2	240.0
4¼	14.186	180.9	102.0	10¼	82.516	2 537.4	246.0
4½	15.904	214.7	108.0	10½	86.590	2 727.6	252.0
4¾	17.721	252.5	114.0	10¾	90.763	2 927.1	258.0
5	19.635	294.5	120.0	11	95.033	3 136.1	264.0
5¼	21.648	340.9	126.0	11¼	99.402	3 354.8	270.0
5½	23.758	392.0	132.0	11½	103.869	3 583.5	276.0
5¾	25.967	447.9	138.0	11¾	108.434	3 822.3	282.0
6	28.274	508.9	144.0	12	113.097	4 071.5	288.0
6¼	30.680	575.2	150.0	12¼	117.858	4 331.3	294.0
6½	33.183	647.1	156.0	12½	122.718	4 601.9	300.0
6¾	35.785	724.6	162.0	12¾	127.677	4 883.6	306.0

NOTE.—1 k. = 1 000 lb.

TABLE 8.—Working Stresses for Columns.

$$p = \frac{16.0\,k}{1 + \dfrac{l^2}{13\,500\,r^2}}$$

$\frac{l}{r}$	p	$\frac{l}{r}$	p	$\frac{l}{r}$	p	$\frac{l}{r}$	p	$\frac{l}{r}$	p
40	14.30 k	68	11.92 k	96	9.51 k	124	7.48 k	185	4.53 k
42	14.15 "	70	11.74 "	98	9.35 "	126	7.35 "	190	4.35 "
44	13.99 "	72	11.56 "	100	9.19 "	128	7.23 "	195	4.19 "
46	13.83 "	74	11.38 "	102	9.04 "	130	7.11 "	200	4.04 "
48	13.67 "	76	11.21 "	104	8.88 "	135	6.81 "	205	3.89 "
50	13.50 "	78	11.03 "	106	8.73 "	140	6.52 "	210	3.75 "
52	13.33 "	80	10.86 "	108	8.58 "	145	6.25 "	215	3.62 "
54	13.16 "	82	10.68 "	110	8.43 "	150	6.00 "	220	3.49 "
56	12.98 "	84	10.51 "	112	8.29 "	155	5.76 "	225	3.37 "
58	12.81 "	86	10.34 "	114	8.15 "	160	5.52 "	230	3.25 "
60	12.63 "	88	10.17 "	116	8.01 "	165	5.30 "	235	3.14 "
62	12.45 "	90	10.00 "	118	7.87 "	170	5.09 "	240	3.04 "
64	12.27 "	92	9.83 "	120	7.74 "	175	4.89 "	245	2.94 "
66	12.10 "	94	9.67 "	122.	7.61 "	180	4.71 "	250	2.84 "

Note.—1 k. = 1 000 lb.

TABLE 9.—Values of Coefficients $\left(1 + \dfrac{l^2}{13\,500\,r^2}\right)$ to Deduce Extreme Fiber Stress in Columns, from the Direct Applied Stress.

$\frac{l}{r}$	Coefficient.	$\frac{l}{r}$	Coefficient.	$\frac{l}{r}$	Coefficient.	$\frac{l}{r}$	Coefficient.	$\frac{l}{r}$	Coefficient.
40	1.118	68	1.343	96	1.683	124	2.139	185	3.535
42	1.131	70	1.363	98	1.711	126	2.176	190	3.674
44	1.143	72	1.384	100	1.741	128	2.214	195	3.817
46	1.157	74	1.406	102	1.771	130	2.252	200	3.963
48	1.171	76	1.428	104	1.801	135	2.350	205	4.113
50	1.185	78	1.451	106	1.832	140	2.452	210	4.267
52	1.200	80	1.474	108	1.864	145	2.558	215	4.424
54	1.216	82	1.498	110	1.896	150	2.667	220	4.585
56	1.232	84	1.523	112	1.929	155	2.780	225	4.750
58	1.249	86	1.548	114	1.963	160	2.896	230	4.919
60	1.267	88	1.574	116	1.997	165	3.017	235	5.091
62	1.285	90	1.600	118	2.032	170	3.141	240	5.267
64	1.303	92	1.627	120	2.067	175	3.269	245	5.446
66	1.323	94	1.655	122	2.103	180	3.400	250	5.630

TABLE 10.—MOMENTS, IN FOOT-KIPS, FOR CLASS E-10 ENGINE LOADING, ONE TRACK.

Class E-10 Engine Loading

Wheel Numbers	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	
Spacing in feet		8	5	5	5	5	6	5	5	5	5	5	5	5	5	5	5	5	
Summations Kips	5.0	15.0	25.0	35.0	45.0	51.5	58.0	64.5	71.0	76.0	86.0	96.0	106.0	116.0	122.5	129.0	135.5	142.0	142.0
Feet	0	8	13	18	23	32	37	43	48	53	64	69	74	79	88	93	99	104	109
Kips	142.0	137.0	127.0	117.0	107.0	97.0	90.5	84.0	77.5	71.0	66.0	56.0	46.0	36.0	26.0	19.5	13.0	6.5	0
Feet	109	101	96	91	86	77	72	66	61	53	45	40	35	30	21	16	10	5	0
End of Train	8132.0	7637.0	6627.0	5667.0	4757.0	3897.0	3396.5	2928.5	2499.5	2103.0	1838.0	1388.0	988.0	638.0	338.0	201.5	97.5	32.5	
18	7472.0	6952.0	5992.0	5082.1	4222.0	3412.0	2944.0	2508.5	2112.0	1748.0	1508.0	1108.0	758.0	458.0	208.0	104.2	32.5		
17	6791.5	6299.5	5389.2	4529.5	3719.5	2959.5	2524.0	2121.0	1757.0	1425.5	1210.5	860.5	560.5	310.5	110.5	39.0	32.5		
16	6020.5	5556.6	4705.5	3906.5	3156.6	2455.5	2059.0	1695.0	1370.0	1077.5	892.5	605.5	362.5	172.5	32.5	39.0	110.5		
15	5408.0	4968.0	4168.0	3418.0	2718.0	2068.0	1704.0	1372.5	1080.0	830.0	660.0	420.0	230.0	90.0	32.5	104.0	208.0		
14	4864.0	3969.0	3259.0	2699.0	1969.0	1429.0	1123.5	850.5	616.5	415.0	300.0	150.0	50.0	58.5	149.5	279.5	442.0		
13	3834.0	3464.0	2804.0	2194.0	1634.0	1124.0	851.0	610.5	409.0	240.0	150.0	50.0	141.0	284.5	427.0	622.0			
12	3354.0	3009.0	2399.0	1839.0	1329.0	869.0	628.5	420.5	251.5	115.0	50.0	50.0	273.5*	429.5	624.5	882.0			
11	2924.0	2604.0	2044.0	1534.0	1074.0	664.0	456.0	280.5	144.0	40.0	50.0	150.0	300.0	456.0	644.5	872.0	1132.0		
10	2316.0	2036.0	1556.0	1126.0	746.0	416.0	260.0	136.5	52.0	80.0	210.0	390.0	620.0	838.0	1068.5	1348.0	1660.0		
9	1748.0	1508.0	1108.0	758.0	458.0	208.0	104.0	32.5	40.0	200.0	410.0	670.0	980.0	1240.0	1532.5	1864.0	2228.0		
8	1425.5	1210.5	860.5	560.5	310.5	110.5	39.0	32.5	97.5	307.5	567.5	877.5	1287.5	1580.0	1865.0	2219.0	2615.5		
7	1077.2	892.5	602.5	362.5	172.5	32.5	39.0	110.5	206.5	475.5	795.5	1165.5	1585.5	1917.0	2281.0	2684.0	3119.5		
6	820.0	660.0	420.0	230.0	90.0	32.5	104.0	208.0	328.0	648.0	1018.0	1438.0	1908.0	2272.0	2668.5	3104.0	3622.0		
5	415.0	300.0	150.0	50.0	58.5	149.5	279.5	442.0	607.0	1017.0	1477.0	1937.0	2547.0	2969.5	3424.5	3918.5	4445.0		
4	240.0	150.0	50.0	141.0	264.5	427.0	622.0	812.0	1272.0	1732.0	2242.0	2952.0	3407.0	3894.5	4421.0	4980.0			
3	115.0	50.0	50.0	273.5	429.5	624.5	852.0	1067.0	1577.0	2187.0	2747.0	3407.0	3894.5	4414.4	4973.5	5565.0			
2	40.0	50.0	150.0	300.0	456.0	644.5	872.0	1132.0	1372.0	1932.0	2542.0	3202.0	3912.0	4432.0	4984.5	5576.0	6200.0		
1	80.0	210.0	390.0	620.0	828.0	1088.5	1348.0	1660.0	1940.0 ~	2690.0	3270.0	4010.0	4800.0	5572.0	5976.5	6620.0	7296.0		

Moments of Wheel Loads About Wheel Number

TABLE 11.—Wheel Determining Maximum Moment for Class E Engine Loading.

Segments in feet	10	15	20	25	30	35	40	45	50	55	60	65	70	80	90	100	110	120	130	140
500 to 260	2	3	3	4	4	5	5	6	7	7	8	9	10	11	12	13	14	15	17	18
250 to 200	2	3	3	4	4	5	5	6	7	8	8	9	10	11	12	13	14	15	17	18
190 to 150	2	3	3	4	4	5	5	6	7	8	9	9	11	12	12	13	14	16	17	18
140	3	3	3	4	4	5	5	6	7	8	9	10	11	12	12	13	14	15	17	18
130	3	3	3	4	4	5	5	6	7	8	9	10	11	12	12	13	14	15	17	
120	3	3	3	4	4	5	6	6	7	8	9	10	11	12	13	13	14	15		
110	3	3	3	4	4	5	6	7	7	8	9	10	11	12	13	13	14			
100	3	3	3	4	5	5	$\overline{14}$	$\overline{14}$	$\overline{14}$	$\overline{14}$	$\overline{13}$	$\overline{13}$	11	12	13	13				
90	3	3	4	4	5	$\overline{13}$	$\overline{13}$	$\overline{13}$	$\overline{13}$	$\overline{13}$	$\overline{13}$	$\overline{13}$	$\overline{13}$	12	13					
80	3	3	4	4	$\overline{13}$	$\overline{13}$	$\overline{13}$	$\overline{12}$	$\overline{12}$	$\overline{12}$	$\overline{12}$	$\overline{12}$	$\overline{12}$	12						
70	3	3	4	4	$\overline{13}$	$\overline{13}$	$\overline{12}$	$\overline{12}$	$\overline{12}$	$\overline{12}$	$\overline{11}$	$\overline{11}$	11							
65	3	3	4	4	$\overline{12}$	$\overline{12}$	$\overline{12}$	$\overline{12}$	$\overline{12}$	$\overline{11}$	$\overline{11}$	11								
60	3	3	4	4	$\overline{13}$	$\overline{13}$	$\overline{12}$	$\overline{11}$	$\overline{11}$	$\overline{11}$	11									
55	12	12	12	4	12	$\overline{13}$	$\overline{12}$	$\overline{12}$	$\overline{13}$	11										
50	12	12	12	12	12	$\overline{13}$	$\overline{13}$	$\overline{13}$	12											
45	12	12	12	12	12	$\overline{13}$	$\overline{13}$	13												
40	3	3	3	12	12	$\overline{13}$	13													
35	3	3	4	4	13	13														
30	3	3	4	4	13															
25	3	3	4	4																
20	$\overline{4}$	3	4																	
15	3	3																		
10	3																			

The shorter segment is ahead followed by the longer one except where the wheel is overlined

Class E Engine

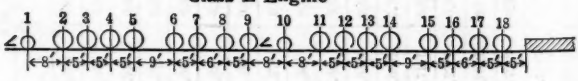
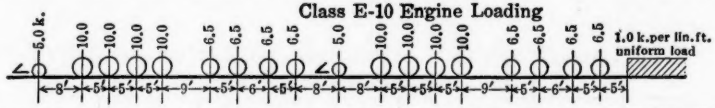


TABLE 12.—Maximum Moments, Shears, and Reactions, Class E-10 Engine Loading, One Track.

Class E-10 Engine Loading



Span, in feet.	Maximum moment, in foot-kips.	Maximum shear, in kips.	Maximum floor-beam reaction, in kips.	Equivalent Uniform Load.			Span, in feet.	Maximum moment, in foot-kips.	Maximum shear, in kips.	Maximum floor-beam reaction, in kips.	Equivalent Uniform Load.		
				Moment.	Shear.	Reaction.					Moment.	Shear.	Reaction.
10	28.1	15.0	20.0	2.25	3.00	2.00	46	414.9	41.4	60.3	1.57	1.80	1.31
11	32.9	16.4	21.9	2.17	2.97	1.99	48	443.8	42.4	62.4	1.54	1.77	1.30
12	40.0	17.5	23.3	2.22	2.92	1.94	50	475.5	43.5	64.3	1.52	1.74	1.29
13	47.5	18.5	24.6	2.25	2.84	1.90	52	507.6	44.6	66.7	1.50	1.72	1.28
14	55.0	19.3	26.1	2.25	2.75	1.86	54	540.5	45.6	69.0	1.48	1.69	1.28
15	62.5	20.0	27.3	2.22	2.67	1.82	56	576.1	46.5	71.4	1.47	1.66	1.27
16	70.0	21.3	28.5	2.19	2.66	1.78	58	611.6	47.7	74.0	1.46	1.65	1.28
17	77.5	22.4	29.4	2.15	2.63	1.73	60	649.5	48.8	76.6	1.44	1.63	1.28
18	85.0	23.3	30.3	2.10	2.59	1.69	62	688.2	50.0	79.1	1.43	1.61	1.27
19	93.3	24.2	31.5	2.07	2.55	1.66	64	727.7	51.3	81.5	1.42	1.60	1.27
20	103.1	25.0	32.8	2.06	2.50	1.64	66	769.7	52.5	83.9	1.41	1.59	1.27
21	112.9	25.7	34.0	2.05	2.45	1.62	68	811.7	53.9	86.2	1.40	1.59	1.27
22	122.8	26.3	35.1	2.03	2.40	1.60	70	853.7	55.3	88.5	1.39	1.58	1.26
23	132.7	27.0	36.1	2.01	2.34	1.57	72	896.7	56.7	90.7	1.38	1.58	1.26
24	142.6	27.7	37.0	1.98	2.31	1.54	74	939.0	58.1	93.0	1.37	1.57	1.26
25	152.5	28.4	37.8	1.95	2.27	1.51	76	986.0	59.5	95.2	1.36	1.57	1.25
26	162.4	29.1	38.8	1.92	2.24	1.49	78	1 032.7	60.9	97.3	1.36	1.56	1.25
27	172.3	29.6	40.0	1.89	2.20	1.48	80	1 080.0	62.1	99.4	1.35	1.55	1.24
28	182.7	30.2	41.2	1.86	2.16	1.47	82	1 128.3	63.5	101.5	1.34	1.55	1.24
29	194.0	30.8	42.2	1.84	2.12	1.46	84	1 177.7	64.8	103.5	1.34	1.54	1.23
30	205.2	31.5	43.1	1.82	2.10	1.44	86	1 229.7	66.1	105.4	1.33	1.54	1.23
31	216.5	32.2	44.3	1.80	2.08	1.43	88	1 282.0	67.4	107.3	1.32	1.53	1.22
32	227.7	32.9	45.5	1.78	2.05	1.42	90	1 334.7	68.6	109.3	1.32	1.53	1.22
33	239.0	33.5	46.7	1.75	2.03	1.41	92	1 388.3	69.9	111.2	1.31	1.52	1.21
34	250.3	34.1	47.8	1.73	2.00	1.40	94	1 442.7	71.2	113.1	1.31	1.52	1.20
35	261.5	34.6	48.8	1.71	1.98	1.39	96	1 497.3	72.4	115.0	1.30	1.51	1.20
36	274.3	35.3	49.8	1.69	1.96	1.38	98	1 552.7	73.7	116.8	1.29	1.51	1.19
37	287.2	35.9	50.7	1.68	1.94	1.37	100	1 609.7	75.0	118.6	1.29	1.50	1.19
38	300.0	36.5	51.8	1.66	1.92	1.36	125	2 497.7	89.7	140.5	1.28	1.44	1.12
39	313.3	37.2	52.9	1.65	1.90	1.36	150	3 531.0	102.7	162.7	1.25	1.38	1.08
40	327.8	37.7	54.0	1.64	1.88	1.35	175	4 676.3	117.3	185.8	1.22	1.34	1.06
42	356.7	39.2	56.0	1.62	1.87	1.34	200	5 939.0	130.5	209.5	1.19	1.31	1.05
44	385.8	40.3	58.2	1.60	1.83	1.32	250	8 796.3	156.6	257.6	1.13	1.25	1.03

Note.—1 k. = 1 000 lb.

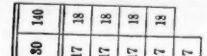
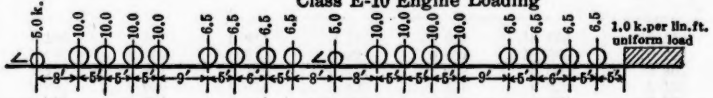


TABLE 15.—(*Continued.*)

Number of panels.	Panel points.	Panel Lengths.										
		13′ 6″	14′ 0″	14′ 6″	15′ 0″	15′ 6″	16′ 0″	16′ 6″	17′ 0″	17′ 6″	18′ 0″	18′ 6″
3	⅙	302	321	340	360	381	403	424	446	468	491	514
	¼	418	446	473	502	530	560	591	621	653	684	716
	¾	541	576	612	648	688	728	770	812	854	896	940
4	⅓	524	558	594	632	672	715	758	804	849	897	942
	⅔	768	819	871	924	983	1 041	1 099	1 158	1 223	1 285	1 354
5	⅙	640	680	738	786	836	888	941	996	1 050	1 108	1 164
	½	980	1 046	1 121	1 196	1 277	1 364	1 451	1 538	1 625	1 723	1 813
	⅚	1 104	1 178	1 255	1 335	1 416	1 497	1 581	1 668	1 767	1 862	1 965
6	⅐	762	819	874	933	992	1 054	1 115	1 178	1 242	1 307	1 374
	⅜	1 224	1 316	1 408	1 504	1 605	1 707	1 808	1 910	2 013	2 113	2 216
	⅝	1 432	1 533	1 644	1 761	1 879	1 999	2 122	2 250	2 382	2 515	2 650
7	⅛	880	943	1 006	1 072	1 138	1 208	1 278	1 349	1 421	1 496	1 572
	⅜	1 466	1 567	1 669	1 774	1 884	1 998	2 112	2 230	2 349	2 472	2 595
	⅝	1 806	1 940	2 076	2 213	2 351	2 491	2 634	2 781	2 931	3 080	3 232
	⅞	1 862	1 999	2 140	2 289	2 432	2 606	2 770	2 929	3 100	3 266	3 442
8	⅛	990	1 060	1 131	1 205	1 278	1 356	1 433	1 514	1 595	1 679	1 764
	⅜	1 690	1 805	1 929	2 043	2 170	2 299	2 430	2 566	2 709	2 843	2 985
	⅝	2 153	2 301	2 451	2 602	2 755	2 909	3 064	3 227	3 400	3 584	3 769
	⅞	2 353	2 536	2 703	2 882	3 063	3 246	3 432	3 619	3 815	4 012	4 213
9	1/10	1 098	1 174	1 253	1 333	1 415	1 500	1 587	1 675	1 766	1 858	1 953
	3/10	1 908	2 037	2 169	2 304	2 445	2 591	2 739	2 891	3 047	3 205	3 367
	5/10	2 451	2 607	2 769	2 946	3 132	3 321	3 514	3 711	3 915	4 125	4 339
	7/10	2 807	2 997	3 194	3 393	3 594	3 798	4 005	4 216	4 477	4 721	4 969
10	9/10	2 891	3 098	3 309	3 528	3 751	3 977	4 206	4 438	4 674	4 918	5 166

TABLE 13.—Maximum Moments, in Foot-Kips, for Truss Bridges, Class E-10 Engine Loading, One Track.

Class E-10 Engine Loading



Panel Lengths.

Number of panels.	Panel points.	8′ 0″	8′ 6″	9′ 0″	9′ 6″	10′ 0″	10′ 6″	11′ 0″	11′ 6″	12′ 0″	12′ 6″	13′ 0″
3	1/3	130	144	157	170	186	201	216	232	248	264	283
4	1/4	173	193	212	233	253	275	297	320	344	367	393
	2/4	226	250	273	299	328	357	386	415	444	476	506
5	1/5	216	240	265	291	318	344	372	400	428	460	490
	2/5	316	351	386	420	460	502	544	587	630	670	717
6	1/6	256	284	314	344	372	407	438	474	512	553	594
	2/6	403	451	491	539	586	635	687	742	799	857	916
	3/6	444	488	540	594	647	707	770	833	896	963	1 032
7	1/7	292	325	358	394	432	474	517	564	612	660	710
	2/7	486	538	591	646	703	762	828	901	976	1 057	1 140
	3/7	570	631	696	764	834	908	993	1 077	1 159	1 244	1 333
8	1/8	328	366	408	453	502	550	600	652	706	760	819
	2/8	561	621	684	749	824	909	996	1 083	1 173	1 266	1 362
	3/8	686	760	840	924	1 012	1 107	1 207	1 313	1 425	1 549	1 678
	4/8	728	812	896	985	1 080	1 192	1 281	1 388	1 497	1 608	1 731
9	1/9	368	416	465	515	567	625	679	738	799	860	924
	2/9	633	706	784	872	962	1 057	1 155	1 256	1 360	1 468	1 578
	3/9	799	886	980	1 080	1 194	1 321	1 451	1 579	1 723	1 865	2 007
	4/9	883	990	1 096	1 206	1 317	1 439	1 573	1 716	2 870	2 027	2 189
10	1/10	414	464	518	574	632	693	755	819	885	953	1 023
	2/10	715	804	897	991	1 093	1 200	1 311	1 425	1 541	1 660	1 783
	3/10	910	1 022	1 139	1 270	1 400	1 544	1 695	1 844	1 993	2 144	2 297
	4/10	1 040	1 156	1 285	1 425	1 577	1 739	1 907	2 077	2 256	2 437	2 621
	5/10	1 080	1 202	1 335	1 470	1 610	1 765	1 930	3 110	3 298	2 490	2 687

TABLE 13.—(*Continued*).

PANEL LENGTHS.

Number of panels.	Panel points.	24′ 6″	25′ 0″	25′ 6″	26′ 0″	26′ 6″	27′ 0″	27′ 6″	28′ 0″	28′ 6″	29′ 0″	29′ 6″
3	1/3	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
4	1/4	1 218	1 265	1 313	1 364	1 415	1 466	1 510	1 566	1 621	1 666	1 720
	2/4	1 553	1 610	1 667	1 731	1 811	1 862	1 943	1 999	2 072	2 147	2 222
5	1/5	1 600	1 660	1 720	1 782	1 844	1 908	1 972	2 037	2 102	2 169	2 236
	2/5	2 346	2 437	2 530	2 621	2 713	2 807	2 900	2 997	3 094	3 194	3 293
6	1/6	1 951	2 024	2 098	2 173	2 349	2 326	2 404	2 483	2 563	2 645	2 727
	2/6	3 010	3 118	3 227	3 341	3 462	3 584	3 707	3 832	3 959	4 087	4 219
	3/6	3 396	3 528	3 661	3 796	3 931	4 068	4 206	4 345	4 483	4 626	4 770
7	1/7	2 288	2 374	2 460	2 549	2 638	2 729	2 820	2 914	3 008	3 105	3 201
	2/7	3 673	3 812	3 950	4 094	4 240	4 392	4 543	4 697	4 850	5 008	5 167
	3/7	4 410	4 578	4 750	4 925	5 101	5 281	5 461	5 645	5 828	6 016	6 208
8	1/8	2 616	2 715	2 814	2 916	3 016	3 122	3 228	3 335	3 443	3 555	3 666
	2/8	4 333	4 498	4 662	4 832	5 003	5 180	5 357	5 540	5 723	5 912	6 100
	3/8	5 396	5 604	5 814	6 025	6 242	6 465	6 687	6 914	7 141	7 372	7 604
	4/8	5 720	5 928	6 136	6 350	6 565	6 786	7 006	7 229	7 454	7 684	7 925
9	1/9	2 928	3 049	3 160	3 275	3 391	3 510	3 628	3 750	3 874	3 998	4 124
	2/9	4 979	5 170	5 362	5 556	5 752	5 955	6 160	6 372	6 584	6 802	7 019
	3/9	6 364	6 611	6 858	7 111	7 366	7 628	7 892	8 162	8 432	8 708	8 984
	4/9	7 024	7 282	7 542	7 806	8 079	8 348	8 623	8 904	9 185	9 471	9 768
10	1/10	3 256	3 379	3 504	3 631	3 761	3 892	4 025	4 160	4 298	4 437	4 579
	2/10	5 618	5 833	6 050	6 271	6 495	6 722	6 956	7 195	7 436	7 681	7 928
	3/10	7 320	7 603	7 891	8 182	8 474	8 778	9 086	9 397	9 711	10 030	10 352
	4/10	8 309	8 616	8 926	9 240	9 564	9 891	10 222	10 556	10 894	11 237	11 593
	5/10	8 468	8 780	9 095	9 417	9 746	10 078	10 416	10 760	11 108	11 463	11 828

TABLE 13.—(*Continued*).

Number of panels.	Panel points.	Panel Lengths.											
		19′ 0″	19′ 6″	20′ 0″	20′ 6″	21′ 0″	21′ 6″	22′ 0″	22′ 6″	23′ 0″	23′ 6″	24′ 0″
3	1/6	538	562	586	....	....	....	....	....	....	....	....
	1/4	747	783	824	866	909	952	996	1 039	1 083	1 128	1 173
4	1/4	986	1 032	1 080	1 128	1 178	1 230	1 282	1 335	1 388	1 443	1 497
	1/6	991	1 040	1 092	1 146	1 200	1 255	1 311	1 367	1 425	1 482	1 541
5	1/5	1 425	1 501	1 577	1 658	1 739	1 822	1 907	1 991	2 077	2 166	2 256
	1/6	1 225	1 284	1 345	1 406	1 471	1 536	1 603	1 670	1 740	1 809	1 880
6	2/6	1 912	2 006	2 104	2 200	2 300	2 399	2 500	2 600	2 702	2 804	2 908
	2/6	2 073	2 185	2 298	2 411	2 528	2 647	2 768	2 891	3 015	3 140	3 266
	1/7	1 442	1 511	1 582	1 652	1 729	1 802	1 881	1 959	2 040	2 121	2 205
7	2/7	2 319	2 422	2 530	2 645	2 766	2 886	3 012	3 138	3 269	3 401	3 537
	3/7	2 785	2 920	3 058	3 196	3 339	3 484	3 632	3 779	3 930	4 086	4 244
	1/8	1 650	1 728	1 810	1 891	1 978	2 060	2 149	2 239	2 332	2 424	2 520
	2/8	2 722	2 850	2 983	3 122	3 265	3 408	3 556	3 704	3 856	4 012	4 172
8	3/8	3 385	3 540	3 694	3 855	4 026	4 211	4 397	4 593	4 790	4 989	5 192
	4/8	3 617	3 796	3 977	4 158	4 345	4 531	4 723	4 918	5 116	5 315	5 518
	1/9	1 852	1 940	2 032	2 123	2 218	2 312	2 412	2 512	2 617	2 722	2 830
	2/9	3 132	3 279	3 431	3 588	3 751	3 916	4 086	4 256	4 433	4 610	4 794
9	3/9	3 987	4 155	4 332	4 535	4 760	4 973	5 191	5 417	5 647	5 882	6 123
	4/9	4 426	4 649	4 869	5 094	5 324	5 555	5 789	6 027	6 274	6 520	6 772
	1/10	2 049	2 148	2 249	2 352	2 456	2 568	2 672	2 788	2 898	3 015	3 134
	2/10	3 531	3 699	3 871	4 045	4 229	4 417	4 608	4 803	5 000	5 201	5 407
	3/10	4 587	4 778	5 008	5 242	5 490	5 721	5 966	6 208	6 491	6 769	7 040
10	4/10	5 220	5 478	5 743	6 011	6 283	6 558	6 837	7 120	7 411	7 707	8 006
	5/10	5 417	5 671	5 928	6 189	6 458	6 731	7 007	7 286	7 568	7 863	8 162

TABLE 13.—(*Continued.*)

Number of panels.	Panel points.	Panel Lengths.										
		35′ 6″	36′ 0″	37′ 0″	38′ 0″	39′ 0″	40′ 0″	41′ 0″	42′ 0″	43′ 0″	44′ 0″	45′ 0″
3	1/3	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	1/4	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
4	2/4	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	1/5	3 196	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
5	2/5	4 598	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	1/6	3 851	3 910	4 109	4 312	4 519	4 730	4 945	5 165	5 397	5 639	5 879
6	2/6	5 961	6 124	6 448	6 777	7 119	7 464	7 806	8 164	8 536	8 894	9 248
	3/6	6 662	6 787	7 119	7 456	7 803	8 164	8 531	8 906	9 287	9 680	10 079
	1/7	4 481	4 598	4 884	5 075	5 323	5 572	5 828	6 087	6 361	6 640	6 925
7	2/7	7 294	7 496	7 895	8 201	8 715	9 136	9 588	10 010	10 460	10 916	11 380
	3/7	8 642	8 866	9 308	9 757	10 229	10 700	11 187	11 695	12 195	12 720	13 251
	1/8	5 142	5 277	5 549	5 838	6 113	6 404	6 701	6 998	7 315	7 688	7 968
8	2/8	8 607	8 828	9 332	9 805	10 297	10 796	11 310	11 837	12 372	12 917	13 470
	3/8	10 684	10 911	11 463	12 025	12 604	13 201	13 810	14 436	15 070	15 726	16 390
	4/8	11 086	11 319	11 901	12 459	13 117	13 744	14 397	15 061	15 741	16 487	17 149
	1/9	5 796	5 948	6 258	6 574	6 898	7 226	7 566	7 911	8 262	8 630	9 004
	2/9	9 918	10 188	10 787	11 296	11 866	12 447	13 039	13 649	14 270	14 908	15 546
9	3/9	12 600	12 934	13 696	14 270	14 965	15 681	16 410	17 161	17 923	18 709	19 540
	4/9	13 662	14 015	14 745	15 495	16 265	17 055	17 865	18 695	19 545	20 415	21 305
	1/10	6 445	6 615	6 961	7 315	7 677	8 048	8 426	8 813	9 207	9 616	10 035
	2/10	11 217	11 519	12 141	12 776	13 434	14 085	14 759	15 450	16 158	16 878	17 610
10	3/10	14 560	14 940	15 712	16 499	17 307	18 142	18 992	19 870	20 759	21 676	22 607
	4/10	16 263	16 692	17 568	18 468	19 392	20 340	21 312	22 308	23 328	24 372	25 440
	5/10	16 708	17 150	18 063	19 000	19 963	20 950	21 963	23 000	24 063	25 150	26 263

TABLE 13.—(*Continued.*)

PANEL LENGTHS.

Number of panels.	Panel points.	30′ 0″	30′ 6″	31′ 0″	31′ 6″	32′ 0″	32′ 6″	33′ 0″	33′ 6″	34′ 0″	34′ 6″	35′ 0″
3	1/3	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
4	1/4	1 774	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	2/4	2 298	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
5	1/5	2 304	2 375	2 445	2 518	2 591	2 672	2 740	2 816	2 891	2 969	3 047
	2/5	3 393	3 494	3 594	3 696	3 798	3 900	4 005	4 118	4 236	4 356	4 477
6	1/6	2 810	2 895	2 980	3 068	3 157	3 248	3 339	3 432	3 525	3 620	3 715
	2/6	4 352	4 488	4 623	4 761	4 899	5 044	5 192	5 342	5 492	5 648	5 804
	3/6	4 918	5 067	5 216	5 367	5 518	5 672	5 825	5 981	6 136	6 298	6 459
7	1/7	3 300	3 400	3 501	3 604	3 706	3 814	3 922	4 032	4 142	4 255	4 368
	2/7	5 332	5 499	5 665	5 836	6 006	6 179	6 354	6 528	6 724	6 914	7 103
	3/7	6 394	6 590	6 786	6 986	7 185	7 390	7 592	7 798	8 006	8 218	8 429
8	1/8	3 780	3 896	4 012	4 130	4 249	4 272	4 496	4 623	4 750	4 880	5 010
	2/8	6 292	6 490	6 688	6 891	7 098	7 301	7 508	7 724	7 941	8 163	8 384
	3/8	7 840	8 082	8 324	8 573	8 820	9 074	9 327	9 584	9 840	10 104	10 368
	4/8	8 162	8 409	8 655	8 907	9 159	9 419	9 680	9 946	10 212	10 486	10 760
9	1/9	4 253	4 384	4 515	4 650	4 784	4 924	5 063	5 207	5 351	5 499	5 646
	2/9	7 240	7 469	7 698	7 933	8 168	8 408	8 645	8 896	9 144	9 401	9 658
	3/9	9 267	9 554	9 841	10 137	10 433	10 736	11 038	11 346	11 654	11 969	12 284
	4/9	10 068	10 371	10 676	10 999	11 312	11 638	11 963	12 296	12 629	12 972	13 316
10	1/10	4 709	4 868	5 039	5 165	5 316	5 468	5 625	5 785	5 947	6 111	6 277
	2/10	8 180	8 435	8 696	8 961	9 229	9 500	9 775	10 052	10 336	10 627	10 920
	3/10	10 678	11 007	11 342	11 684	12 030	12 380	12 733	13 089	13 450	13 811	14 183
	4/10	11 952	12 316	12 688	13 065	13 444	13 831	14 226	14 623	15 025	15 435	15 850
	5/10	12 200	12 578	12 963	13 353	13 750	14 154	14 563	14 978	15 400	15 828	16 263

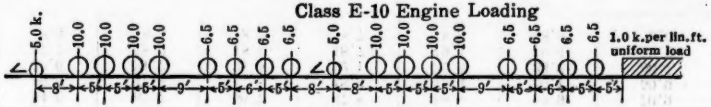
TABLE 14.—(*Continued.*)

PANEL LENGTHS.

Number of panels.	Panel point.	13′ 6″	14′ 0″	14′ 6″	15′ 0″	15′ 6″	16′ 0″	16′ 6″	17′ 0″	17′ 6″	18′ 0″	18′ 6″
3	1/3	22.4	23.0	23.5	24.0	24.6	25.2	25.7	26.2	26.8	27.3	27.8
	2/3	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
4	1/4	31.0	31.8	32.6	33.4	34.2	34.9	35.6	36.3	37.0	37.8	38.6
	2/4	15.1	15.4	15.8	16.2	16.7	17.1	17.6	18.0	18.4	18.9	19.3
	3/4	3.8	3.9	4.1	4.3	4.5	4.7	4.9	5.1	5.2	5.4	5.6
5	1/5	38.6	39.7	40.9	42.2	43.4	44.7	46.0	47.3	48.6	49.8	51.0
	2/5	23.5	24.1	24.8	25.4	25.9	26.5	27.1	27.7	28.3	29.0	29.6
	3/5	11.5	11.8	12.2	12.5	12.8	13.1	13.4	13.7	14.0	14.3	14.6
6	1/6	47.5	49.2	50.8	52.4	54.0	55.5	57.1	58.6	60.1	61.5	63.0
	2/6	31.2	31.9	32.9	33.8	34.8	36.0	37.2	38.3	39.4	40.4	41.4
	3/6	19.1	19.6	20.2	20.7	21.2	21.6	22.1	22.6	23.0	23.4	23.8
	4/6	9.0	9.3	9.6	9.9	10.2	10.5	10.8	11.0	11.3	11.6	11.8
7	1/7	56.6	58.5	60.4	62.2	64.0	65.8	67.6	69.3	71.0	72.6	74.3
	2/7	39.5	41.0	42.4	43.8	45.2	46.6	47.9	49.2	50.5	51.8	53.1
	3/7	26.4	27.0	27.7	28.4	29.2	30.1	31.0	31.9	32.8	33.7	34.6
	4/7	15.9	16.4	16.9	17.4	17.8	18.2	18.6	19.0	19.4	19.8	20.2
	5/7	7.4	7.6	7.9	8.1	8.4	8.6	8.9	9.1	9.4	9.6	9.8
8	1/8	65.2	67.4	69.4	71.5	73.5	75.5	77.4	79.3	81.2	83.1	85.0
	2/8	48.4	50.1	51.8	53.5	55.1	56.7	58.3	59.8	61.3	62.8	64.2
	3/8	34.0	35.1	36.4	37.6	38.7	39.8	41.0	42.2	43.4	44.6	45.7
	4/8	22.7	23.2	23.9	24.6	25.2	25.9	26.6	27.4	28.2	28.9	29.6
	5/8	13.6	14.0	14.4	14.8	15.2	15.6	16.0	16.4	16.7	17.0	17.4
	6/8	6.0	6.3	6.6	6.8	7.0	7.2	7.5	7.7	7.9	8.1	8.3
9	1/9	73.4	75.8	78.0	80.3	82.5	84.7	86.9	89.1	91.2	93.3	95.4
	2/9	57.0	59.0	60.8	62.7	64.5	66.3	68.0	69.8	71.5	73.2	74.9
	3/9	42.6	43.9	45.2	46.7	48.2	49.6	51.0	52.4	53.8	55.1	56.4
	4/9	29.9	30.9	32.0	33.1	34.1	35.0	36.0	37.0	38.0	39.0	40.0
	5/9	19.8	20.3	21.0	21.5	22.2	22.8	23.4	24.1	24.8	25.4	26.1
	6/9	11.7	12.1	12.6	12.9	13.2	13.6	13.9	14.3	14.6	14.9	15.2
	7/9	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
10	1/10	81.3	83.9	86.4	88.9	91.3	93.8	96.2	98.5	100.9	103.3	105.6
	2/10	65.3	67.4	69.5	71.5	73.6	75.6	77.5	79.5	81.4	83.3	85.2
	3/10	50.9	52.6	54.2	55.7	57.3	59.0	60.6	62.1	63.7	65.2	66.8
	4/10	38.0	39.3	40.6	41.8	43.0	44.2	45.4	46.6	47.7	48.9	50.1
	5/10	28.6	27.5	28.5	29.5	30.4	31.3	32.2	33.1	34.0	34.8	35.7
	6/10	17.6	18.1	18.6	19.1	19.7	20.3	20.8	21.4	22.1	22.7	23.3
	7/10	10.2	10.6	11.0	11.3	11.7	12.0	12.3	12.6	12.9	13.3	13.5
	8/10	4.4	4.6	4.7	4.9	5.1	5.3	5.5	5.7	5.9	6.1	6.3

TABLE 14.—Maximum Shears, in Kips, for Truss Bridges, Class E-10 Engine-Loading, One Track.

Class E-10 Engine Loading



Panel Lengths.

Number of panels.	Panel point.	8′ 0″	8′ 6″	9′ 0″	9′ 6″	10′ 0″	10′ 6″	11′ 0″	11′ 6″	12′ 0″	12′ 6″	13′ 0″
3	1/3	16.2	16.8	17.4	17.9	18.6	19.2	19.7	20.2	20.6	21.2	21.7
	2/3	2.9	3.2	3.5	3.8	4.0	4.4	4.7	5.0	...	...	...
4	1/4	21.6	22.7	23.6	24.5	25.2	26.2	27.0	27.8	28.6	29.4	30.2
	2/4	9.4	10.2	10.8	11.4	12.0	12.5	13.0	13.4	13.8	14.2	14.7
	3/4	1.3	1.4	1.5	1.8	2.0	2.3	2.6	2.9	3.2	3.4	3.6
5	1/5	26.9	28.1	29.4	30.6	31.8	32.8	33.8	34.8	35.7	36.6	37.4
	2/5	15.5	16.4	17.2	18.0	18.7	19.5	20.1	20.8	21.5	22.2	22.8
	3/5	6.5	7.2	7.8	8.3	8.8	9.2	9.6	10.0	10.4	10.8	11.1
6	1/6	32.0	33.4	34.8	36.0	37.4	38.8	40.0	41.2	42.7	44.2	45.7
	2/6	21.1	22.1	23.2	24.2	25.2	26.2	27.1	28.0	28.8	29.7	30.5
	3/6	12.1	12.7	13.6	14.2	15.0	15.6	16.2	16.8	17.4	18.0	18.5
	4/6	4.6	5.2	5.8	6.3	6.6	7.1	7.4	7.8	8.1	8.4	8.8
7	1/7	36.4	38.0	39.7	41.4	43.2	45.1	47.0	49.2	51.0	52.8	54.6
	2/7	26.2	27.6	29.0	30.1	31.3	32.4	33.6	34.4	35.6	36.8	38.0
	3/7	17.4	18.2	19.2	20.2	21.0	21.9	22.8	23.5	23.8	24.8	25.7
	4/7	9.6	10.4	11.0	11.6	12.2	12.8	13.4	13.9	14.4	15.0	15.4
	5/7	.....	.....	.....	.....	.....	.....	.....	.....	6.4	6.8	7.1
8	1/8	40.8	43.0	45.4	47.7	50.2	52.4	54.6	56.8	58.9	60.9	63.0
	2/8	31.3	32.7	34.2	35.6	37.0	38.4	39.9	41.6	43.4	45.0	46.7
	3/8	22.3	23.6	24.8	25.8	26.9	27.8	28.9	29.8	30.7	31.8	32.9
	4/8	14.6	15.4	16.3	17.1	17.8	18.7	19.4	20.2	20.8	21.5	22.1
	5/8	7.8	8.5	9.1	9.6	10.2	10.8	11.2	11.6	12.2	12.7	13.1
	6/8	.....	.....	.....	.....	.....	.....	.....	.....	5.2	5.5	5.8
9	1/9	46.1	48.9	51.7	54.2	56.8	59.2	61.7	64.3	66.6	68.8	71.0
	2/9	35.8	37.4	39.4	41.3	43.3	45.4	47.4	49.4	51.3	53.2	55.0
	3/9	27.2	28.6	29.9	31.0	32.5	33.7	35.1	36.6	38.2	39.7	41.2
	4/9	19.3	20.4	21.5	22.6	23.4	24.7	25.2	26.0	27.0	27.9	28.9
	5/9	12.4	13.2	14.0	14.8	15.4	16.2	16.9	17.5	18.1	18.7	19.3
	6/9	.....	.....	.....	.....	.....	.....	.....	.....	10.5	10.9	11.3
	7/9	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
10	1/10	51.7	54.6	57.6	60.4	63.2	66.0	68.7	71.2	73.8	76.3	78.8
	2/10	40.5	42.9	45.3	47.8	50.0	52.3	54.6	56.8	58.9	61.1	63.2
	3/10	31.5	33.3	35.0	36.7	38.6	40.5	42.4	44.1	45.8	47.5	49.2
	4/10	24.0	25.2	26.4	27.5	28.7	30.0	31.2	32.6	34.0	35.4	36.7
	5/10	16.8	17.9	18.9	19.8	20.7	21.5	22.3	23.1	23.9	24.8	25.7
	6/10	10.6	11.3	12.1	12.8	13.5	14.2	14.8	15.4	16.0	16.6	17.1
	7/10	5.8	6.2	6.6	7.0	7.4	7.8	8.3	8.7	9.1	9.5	9.9
	8/10	2.3	2.5	2.7	2.9	3.1	3.3	3.5	3.7	3.8	4.0	4.2

TABLE 14.—(*Continued.*)

	7/10	13.8	14.1	14.4	14.6	14.9	15.2	15.4	15.7	15.9	16.2	16.4
	8/10	6.4	6.6	6.7	6.9	7.1	7.3	7.4	7.6	7.7	7.9	8.0

TABLE 14.—(*Continued*)

Number of panels.	Panel point.	Panel Lengths. 24′ 6″	25′ 0″	25′ 6″	26′ 0″	26′ 6″	27′ 0″	27′ 6″	28′ 0″	28′ 6″	29′ 0″	29′ 6″
3	1/3	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	2/3	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	3/3	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
4	1/4	49.8	50.6	51.5	52.4	53.2	54.1	54.9	55.7	56.6	57.4	58.3
	2/4	23.6	23.9	24.2	24.5	24.8	25.2	25.5	25.8	26.2	26.6	27.0
	3/4	7.0	7.1	7.2	7.3	7.4	7.5	7.6	7.7	7.8	7.9	8.0
5	1/5	65.3	66.4	67.5	68.6	69.6	70.7	71.8	72.8	73.8	74.8	75.8
	2/5	38.0	38.6	39.3	40.0	40.7	41.4	42.1	42.8	43.5	44.2	44.9
	3/5	18.0	18.2	18.5	18.8	19.1	19.3	19.6	19.9	20.2	20.5	20.8
6	1/6	79.6	81.0	82.3	83.6	84.9	86.2	87.4	88.7	90.0	91.2	92.4
	2/6	52.9	53.8	54.7	55.6	56.5	57.4	58.3	59.2	60.1	61.0	61.8
	3/6	31.0	31.5	32.1	32.6	33.2	32.7	34.3	34.8	35.3	35.8	36.4
	4/6	14.6	14.8	15.0	15.2	15.4	15.6	14.8	16.0	16.2	16.4	16.6
7	1/7	93.4	95.0	96.6	98.1	99.6	101.1	102.6	104.1	105.6	107.1	108.6
	2/7	67.0	68.1	69.3	70.4	71.5	72.6	73.7	74.8	75.9	77.0	78.1
	3/7	44.7	45.4	46.2	47.0	47.7	48.4	49.2	49.9	50.6	51.3	52.0
	4/7	25.9	26.3	26.8	27.3	27.8	28.3	28.8	29.2	29.7	30.2	30.7
	5/7	12.3	12.5	12.7	12.8	13.0	13.2	13.4	13.5	13.7	13.8	14.0
8	1/8	106.8	108.6	110.4	112.2	114.0	115.7	117.4	119.2	120.9	122.6	124.3
	2/8	80.7	82.0	83.3	84.6	86.0	87.4	88.7	90.0	91.4	92.7	94.0
	3/8	58.1	59.0	60.0	60.9	61.9	62.8	63.8	64.7	65.6	66.5	67.4
	4/8	38.5	39.2	39.9	40.6	41.2	41.8	42.5	43.2	43.8	44.4	45.0
	5/8	22.1	22.6	23.0	23.4	23.8	24.2	24.6	25.0	25.5	25.9	26.4
	6/8	10.6	10.8	10.9	11.0	11.2	11.4	11.5	11.6	11.8	12.0	12.2
9	1/9	120.0	122.0	124.0	126.0	128.0	130.0	132.0	134.0	136.0	137.9	139.9
	2/9	94.0	95.5	97.1	98.7	100.2	101.8	103.4	105.0	106.5	108.1	109.6
	3/9	71.2	72.3	73.5	74.7	75.8	77.0	78.1	79.2	80.4	81.5	82.6
	4/9	51.1	52.0	52.8	53.6	54.5	55.4	56.2	57.0	57.8	58.6	59.4
	5/9	33.7	34.3	35.0	35.6	36.2	36.7	37.3	37.9	38.5	39.0	39.6
	6/9	19.4	19.8	20.2	20.6	21.0	21.3	21.7	22.0	22.4	22.7	23.0
	7/9	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
10	1/10	132.9	135.2	137.4	139.7	141.9	144.1	146.4	148.6	150.8	153.0	155.2
	2/10	107.1	108.8	110.7	112.5	114.3	116.1	117.9	119.7	121.5	123.3	125.0
	3/10	84.1	85.5	86.9	88.2	89.6	90.9	92.3	93.7	95.1	96.4	97.8
	4/10	63.6	64.6	65.7	66.7	67.8	68.8	69.9	70.9	71.9	72.9	74.0
	5/10	45.5	46.2	47.0	47.8	48.6	49.4	50.1	50.9	51.7	52.4	53.1
	6/10	30.1	30.6	31.7	31.6	32.1	32.6	33.2	33.7	34.2	34.7	35.3
	7/10	17.3	17.7	18.0	18.3	18.7	19.1	19.4	19.7	20.0	20.3	20.6
	8/10	8.1	8.3	8.4	8.5	8.7	8.8	8.9	9.0	9.2	9.3	9.5

TABLE 14.—(*Continued.*)

Number of panels.	Panel point.	Panel Lengths. 19′ 0″	19′ 6″	20′ 0″	20′ 6″	21′ 0″	21′ 6″	22′ 0″	22′ 6″	23′ 0″	23′ 6″	24′ 0″
3	1/3	28.3	28.8	.....	.....	.....	.....	.....	.....	.....	.....	.....
	2/3	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
4	1/4	39.3	40.3	41.2	42.2	43.3	44.3	45.3	46.2	47.1	48.0	48.9
	2/4	19.7	20.1	20.5	20.9	21.2	21.6	22.0	22.3	22.7	23.0	23.3
	3/4	5.7	5.9	6.0	6.1	6.2	6.4	6.5	6.6	6.7	6.8	6.9
5	1/5	52.2	53.4	54.6	55.9	57.2	58.4	59.6	60.8	62.0	63.1	64.2
	2/5	30.3	31.0	31.6	32.4	33.0	33.8	34.4	35.2	36.0	36.7	37.4
	3/5	14.9	15.2	15.5	15.8	16.1	16.4	16.6	16.9	17.2	17.5	17.7
6	1/6	64.4	65.8	67.2	68.7	70.1	71.5	72.9	74.3	75.6	77.0	78.3
	2/6	42.4	43.4	44.4	45.4	46.4	47.4	48.3	49.3	50.2	51.1	52.0
	3/6	24.3	24.8	25.4	26.0	26.6	27.3	27.9	28.5	29.2	29.8	30.4
	4/6	12.1	12.4	12.6	12.8	13.1	13.4	13.6	13.8	14.0	14.2	14.4
7	1/7	75.9	77.6	79.1	80.7	82.2	83.8	85.5	87.2	88.7	90.3	91.9
	2/7	54.4	55.6	56.8	58.0	59.2	60.4	61.5	62.4	63.7	64.8	65.9
	3/7	35.5	36.4	37.2	38.1	39.0	39.8	40.6	41.5	42.3	43.2	43.9
	4/7	20.6	21.0	21.4	21.8	22.2	22.7	23.2	23.7	24.2	24.8	25.4
	5/7	10.0	10.3	10.5	10.8	11.0	11.2	11.4	11.6	11.8	12.0	12.1
8	1/8	86.8	88.7	90.5	92.3	94.1	95.9	97.7	99.6	101.4	103.2	105.0
	2/8	65.6	67.1	68.5	69.9	71.2	72.6	74.0	75.4	76.7	78.0	79.3
	3/8	46.8	47.9	49.0	50.0	51.0	52.1	53.2	54.2	55.1	56.1	57.1
	4/8	30.3	31.1	31.9	32.7	33.4	34.2	34.9	35.6	36.4	37.1	37.8
	5/8	17.7	18.1	18.4	18.8	19.2	19.6	20.0	20.4	20.8	21.2	21.6
	6/8	8.5	8.8	9.0	9.2	9.4	9.6	9.8	10.0	10.1	10.3	10.4
9	1/9	97.4	99.5	101.6	103.6	105.6	107.6	109.6	111.7	113.8	115.9	118.0
	2/9	76.5	78.2	79.8	81.4	83.0	84.6	86.2	87.8	89.3	90.9	92.4
	3/9	57.7	59.0	60.2	61.5	62.8	64.0	65.2	66.4	67.6	68.8	70.0
	4/9	41.0	42.0	42.9	43.9	44.8	45.7	46.6	47.6	48.5	49.4	50.2
	5/9	26.8	27.4	28.0	28.7	29.3	30.0	30.6	31.2	37.8	32.5	33.1
	6/9	15.5	15.8	16.2	16.5	16.8	17.2	17.6	18.0	18.3	18.7	19.0
	7/9	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
10	1/10	107.9	110.1	112.4	114.7	117.0	119.3	121.5	123.8	126.0	128.3	130.6
	2/10	87.1	89.0	90.8	92.7	94.5	96.4	98.2	100.0	101.7	103.5	105.3
	3/10	68.3	69.8	71.2	72.7	74.1	75.6	77.1	78.5	79.9	81.3	82.7
	4/10	51.3	52.5	53.6	54.8	55.9	57.0	58.2	59.3	60.3	61.4	62.5
	5/10	36.5	37.4	38.2	39.0	39.8	40.6	41.4	42.3	43.1	43.9	44.7
	6/10	23.9	24.5	25.0	25.6	26.2	26.8	27.3	27.9	28.4	29.0	29.5
	7/10	13.8	14.1	14.4	14.7	15.0	15.3	15.6	15.9	16.3	16.6	17.0
	8/10	6.4	6.6	6.7	6.9	7.1	7.3	7.4	7.6	7.7	7.9	8.0

TABLE 14.—(*Continued*)

	7/10	20.9	21.2	21.5	21.8	22.1	22.4	22.7	23.0	23.3	23.6	23.9
	8/10	9.6	9.7	9.8	10.0	10.1	10.3	10.4	10.6	10.7	10.9	11.0

TABLE 14.—(*Continued*).

Number of panels.	Panel point.	Panel Lengths. 35′ 6″	36′ 0″	37′ 0″	38′ 0″	39′ 0″	40′ 0″	41′ 0″	42′ 0″	43′ 0″	44′ 0″	45′ 0″
3	1/3	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	2/3	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
4	1/4	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	2/4	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	3/4	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
5	1/5	88.1	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	2/5	52.4	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	3/5	24.7	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
6	1/6	107.4	108.4	111.0	113.5	115.9	118.2	120.6	123.0	125.5	128.0	130.5
	2/6	71.8	72.6	74.1	75.7	77.3	78.8	80.5	82.1	83.7	85.3	86.9
	3/6	42.6	43.1	44.1	45.1	46.0	47.0	48.0	48.9	49.8	50.8	51.7
	4/6	20.0	20.2	20.7	21.2	21.7	22.2	22.7	23.2	23.7	24.2	24.6
7	1/7	126.3	127.7	130.7	133.6	136.4	139.3	142.2	145.0	147.9	150.9	153.9
	2/7	90.6	91.7	93.7	95.7	97.8	99.9	101.9	104.0	106.0	108.1	110.1
	3/7	60.5	61.2	62.6	64.0	65.4	66.7	68.1	69.4	70.7	72.0	73.3
	4/7	35.9	36.3	37.1	38.0	38.8	39.5	40.3	41.1	41.9	42.7	43.5
	5/7	16.6	16.9	17.3	17.8	18.2	18.6	19.1	19.5	19.9	20.3	20.7
8	1/8	144.9	146.6	150.0	153.4	156.7	160.1	163.4	166.8	170.2	173.6	177.1
	2/8	109.2	110.5	113.0	115.5	118.0	120.6	123.1	125.6	128.1	130.6	133.1
	3/8	78.5	79.4	81.2	83.0	84.7	86.5	88.3	90.0	91.7	93.5	95.2
	4/8	52.4	52.9	54.1	55.2	56.4	57.6	58.8	60.0	61.2	62.3	63.5
	5/8	31.0	31.4	32.1	32.8	33.5	34.2	34.9	35.6	36.3	37.0	37.7
	6/8	14.2	14.4	14.8	15.2	15.6	15.9	16.3	16.7	17.1	17.5	17.8
9	1/9	163.3	165.2	169.1	173.0	176.9	180.7	184.5	188.4	192.2	196.1	200.1
	2/9	127.7	129.2	132.1	135.1	138.1	141.1	144.1	147.1	150.1	153.1	156.0
	3/9	96.3	97.5	99.7	101.9	104.1	106.3	108.5	110.7	112.9	115.0	117.2
	4/9	69.0	69.8	71.4	73.0	74.6	76.2	77.8	79.4	80.9	82.5	84.0
	5/9	46.2	46.7	47.7	48.7	49.8	50.8	51.8	52.8	53.8	54.8	55.8
	6/9	27.1	27.5	28.2	28.9	29.5	30.1	30.8	31.4	32.0	32.6	33.2
	7/9	.....	12.7	13.1	13.4	13.7	14.0	14.3	14.6	14.9	15.3	15.6
10	1/10	181.5	183.7	188.5	192.5	196.9	201.2	205.5	209.8	214.2	218.6	222.9
	2/10	146.0	147.7	151.1	154.6	158.1	161.6	165.1	168.5	172.0	175.4	178.9
	3/10	114.2	115.6	118.2	120.9	123.6	126.2	128.8	131.5	134.2	136.8	139.5
	4/10	86.1	87.1	89.1	91.1	93.1	95.1	97.1	99.1	101.1	103.0	105.0
	5/10	61.8	62.5	63.9	65.3	66.7	68.1	69.5	70.9	72.3	73.7	75.1
	6/10	41.2	41.7	42.6	43.6	44.5	45.4	46.4	47.3	48.2	49.1	50.0
	7/10	24.2	24.4	25.0	25.6	26.3	26.8	27.4	28.0	28.5	29.1	29.6
	8/10	11.2	11.3	11.7	12.0	12.3	12.5	12.8	13.1	13.4	13.7	14.0

TABLE 14.—(*Continued*).

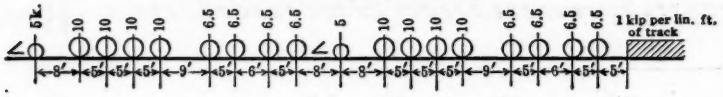
Number of panels.	Panel point.	Panel Lengths. 30′ 0″	30′ 6″	31′ 0″	31′ 6″	32′ 0″	32′ 6″	33′ 0″	33′ 6″	34′ 0″	34′ 6″	35′ 0″
3	1/3	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	2/3	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
4	1/4	59.2	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	2/4	27.3	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	3/4	8.1	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
5	1/5	76.8	77.8	78.8	79.9	81.0	82.0	83.0	84.0	85.0	86.0	87.0
	2/5	45.6	46.2	46.9	47.6	48.2	48.8	49.4	50.0	50.6	51.2	51.8
	3/5	21.1	21.4	21.7	22.0	22.3	22.7	23.0	23.3	23.6	24.0	24.3
6	1/6	93.7	94.9	96.1	97.4	98.6	99.9	101.2	102.4	103.6	104.9	106.2
	2/6	62.7	63.5	64.4	65.2	66.0	66.9	67.7	68.6	69.4	70.2	71.0
	3/6	37.0	37.5	38.0	38.5	39.0	39.5	40.0	40.5	41.0	41.5	42.0
	4/6	16.9	17.2	17.5	17.8	18.0	18.3	18.6	18.9	19.2	19.4	19.7
7	1/7	110.0	111.5	112.9	114.4	115.8	117.3	118.8	120.3	121.8	123.3	124.8
	2/7	79.2	80.2	81.3	82.4	83.4	84.5	85.5	86.6	87.6	88.6	89.6
	3/7	52.7	53.4	54.1	54.8	55.6	56.3	57.0	57.7	58.4	59.1	59.8
	4/7	31.1	31.6	32.0	32.5	33.0	33.4	33.8	34.2	34.6	35.1	35.5
	5/7	14.2	14.4	14.6	14.8	15.0	15.2	15.4	15.7	16.0	16.2	16.4
8	1/8	126.0	127.7	129.4	131.1	132.8	134.5	136.2	138.0	139.7	141.5	143.2
	2/8	95.3	96.6	97.8	99.1	100.4	101.7	103.0	104.2	105.5	106.8	108.0
	3/8	68.3	69.2	70.2	71.1	72.1	73.0	73.9	74.8	75.8	76.7	77.6
	4/8	45.6	46.3	46.9	47.5	48.1	48.8	49.4	50.0	50.6	51.2	51.8
	5/8	26.8	27.2	27.6	28.0	28.4	28.8	29.2	29.6	29.9	30.3	30.6
	6/8	12.3	12.5	12.6	12.8	13.0	13.2	13.3	13.5	13.7	13.9	14.
9	1/9	141.8	143.7	145.6	147.6	149.6	151.5	153.4	155.4	157.4	159.4	161.3
	2/9	111.1	112.6	114.2	115.7	117.2	118.7	120.2	121.7	123.2	124.7	126.2
	3/9	83.8	85.0	86.1	87.3	88.4	89.6	90.7	91.8	93.0	94.1	95.2
	4/9	60.2	61.1	61.9	62.7	63.5	64.3	65.0	65.8	66.6	67.4	68.2
	5/9	40.2	40.7	41.2	41.8	42.4	42.9	43.4	44.0	44.6	45.1	45.6
	6/9	23.4	23.8	24.1	24.5	24.8	25.2	25.5	25.9	26.2	26.5	26.8
	7/9	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
10	1/10	157.4	159.6	161.8	164.0	166.2	168.4	170.6	172.8	174.9	177.1	179.3
	2/10	126.8	128.6	130.3	132.1	133.8	135.6	137.3	139.0	140.8	142.5	144.2
	3/10	99.2	100.6	102.0	103.3	104 7	106.1	107.5	108.8	110.2	111.5	112.9
	4/10	75.0	76.0	77.0	78.0	79.0	80.0	81.0	82.0	83.0	84.0	85.0
	5/10	53.9	54.6	55.3	56.1	56.8	57.5	58.2	58.9	59.6	60.4	61.1
	6/10	35.8	36.3	36.8	37.3	37.8	38.3	38.8	39.3	39.7	40.2	40.7
	7/10	20.9	21.2	21.5	21.8	22.1	22.4	22.7	23.0	23.3	23.6	23.9
	8/10	9.6	9.7	9.8	10.0	10.1	10.3	10.4	10.6	10.7	10.9	11.0

pers. TABLE 14.—(*Continued*).

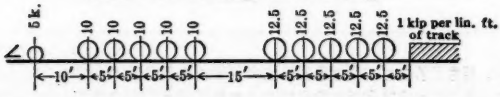
TABLE 15.—CONVERSION TABLE, CLASS E TO CLASS M ENGINE LOADING.

To Convert E-Rating to Equivalent M-Rating Multiply by the Coefficients in this Table
To Convert M-Rating to Equivalent E-Rating Divide by the Coefficients in this Table

Class E-10 Engine Loading



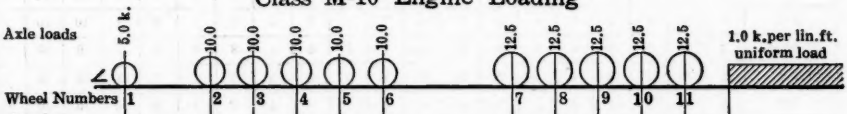
Class M-10 Engine Loading



Span length, in feet	Maximum moment	Maximum shear	Maximum floor beam reaction	Span length, in feet	Maximum moment	Maximum shear	Maximum floor beam reaction
10	0.800	0.800	0.800	46	0.755	0.771	0.777
11	0.801	0.799	0.769	48	0.759	0.771	0.777
12	0.800	0.800	0.747	50	0.765	0.775	0.778
13	0.800	0.800	0.732	52	0.769	0.772	0.782
14	0.801	0.799	0.730	54	0.769	0.767	0.786
15	0.800	0.800	0.729	56	0.774	0.760	0.793
16	0.800	0.800	0.727	58	0.777	0.759	0.803
17	0.801	0.800	0.725	60	0.779	0.756	0.812
18	0.800	0.800	0.724	62	0.781	0.756	0.818
19	0.800	0.799	0.729	64	0.782	0.759	0.825
20	0.800	0.800	0.739	66	0.785	0.760	0.831
21	0.300	0.786	0.746	68	0.785	0.766	0.837
22	0.786	0.773	0.754	70	0.786	0.770	0.845
23	0.772	0.763	0.757	72	0.784	0.775	0.850
24	0.760	0.760	0.761	74	0.781	0.784	0.857
25	0.751	0.758	0.764	76	0.782	0.790	0.862
26	0.743	0.756	0.765	78	0.778	0.795	0.867
27	0.735	0.752	0.770	80	0.776	0.801	0.873
28	0.731	0.749	0.774	82	0.775	0.808	0.877
29	0.729	0.746	0.778	84	0.775	0.815	0.881
30	0.730	0.748	0.778	86	0.775	0.825	0.885
31	0.729	0.750	0.779	88	0.774	0.828	0.887
32	0.728	0.752	0.782	90	0.774	0.839	0.893
33	0.726	0.751	0.786	92	0.774	0.850	0.895
34	0.726	0.752	0.788	94	0.777	0.854	0.898
35	0.724	0.751	0.789	96	0.777	0.855	0.901
36	0.726	0.754	0.786	98	0.777	0.859	0.906
37	0.728	0.753	0.782	100	0.776	0.858	0.909
38	0.731	0.757	0.781	125	0.823	0.874	0.937
39	0.732	0.758	0.782	150	0.859	0.883	0.953
40	0.738	0.759	0.782	175	0.886	0.893	0.964
42	0.745	0.768	0.779	200	0.905	0.902	0.971
44	0.752	0.768	0.776	250	0.939	0.918	0.981

TABLE 16.—MOMENTS, IN FOOT-KIPS, FOR CLASS M-10 ENGINE LOADING.

Class M-10 Engine Loading



Summations													
	Kips	5.0	15.0	25.0	35.0	45.0	55.0	67.5	80.0	92.5	105.0	117.5	117.5
	Feet	0	10	15	20	25	30	45	50	55	60	65	70
	Kips	117.5	112.5	102.5	92.5	82.5	72.5	62.5	50.0	37.5	25.0	12.5	0
	Feet	70	60	55	50	45	40	25	20	15	10	5	0

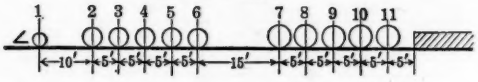
Moments of Wheel Loads About Wheel Number												
End of Train		3787.5	3437.5	2837.5	2287.5	1787.5	1337.5	937.5	625.0	375.0	187.5	62.5
	11	3200.0	2875.0	2325.0	1825.0	1375.0	975.0	625.0	375.0	187.5	62.5	
	10	2675.0	2375.0	1875.0	1425.0	1025.0	675.0	375.0	187.5	62.5	62.5	
	9	2212.5	1937.5	1487.5	1087.5	737.5	437.5	187.5	62.5	62.5	187.5	
	8	1812.5	1562.5	1162.5	812.5	512.5	262.5	62.5	62.5	187.5	375.0	
	7	1475.0	1250.0	900.0	600.0	350.0	150.0	62.5	187.5	375.0	625.0	
	6	650.0	500.0	300.0	150.0	50.0	187.5	437.5	750.0	1125.0	1562.5	
	5	425.0	300.0	150.0	50.0	50.0	300.0	612.5	987.5	1425.0	1925.0	
	4	250.0	150.0	50.0	50.0	150.0	462.5	837.5	1275.0	1775.0	2337.5	
	3	125.0	50.0	50.0	150.0	300.0	675.0	1112.5	1612.5	2175.0	2800.0	
	2	50.0	50.0	150.0	300.0	500.0	937.5	1437.5	2000.0	2625.0	3312.5	
	1	100.0	250.0	450.0	700.0	1000.0	1562.5	2187.5	2875.0	3625.0	4437.5	

TABLE 17.—Wheel Determining Maximum Moment for Class M Engine Loading.

Segments in feet	5	10	15	20	25	30	35	40	45	50	55	60	65	70	75	80	85	90	100	110
1000 to 300	2	3	3	3	4	4	5	5	6	6	7	7	8	8	9	9	9	10	10	11
290 to 250	2	3	3	3	4	4	5	5	6	7	7	7	8	8	9	9	9	10	10	11
240	2	3	3	3	4	4	5	6	6	7	7	7	8	8	9	9	9	10	10	11
230	2	3	3	3	4	4	5	6	6	7	7	7	8	8	9	9	9	10	10	11
220	2	3	3	3	4	4	5	6	6	7	7	8	8	8	9	9	9	10	10	11
210	2	3	3	3	4	4	5	6	6	7	7	8	8	8	9	9	9	10	10	11
200 to 160	2	3	3	3	4	5	5	6	6	7	7	8	8	8	9	9	9	10	10	11
150	2	3	3	3	4	5	5	6	6	7	7	8	8	8	9	9	9	10	10	11
140	2	3	3	4	4	5	5	6	6	7	7	8	8	9	9	9	10	10	10	11
130	2	3	3	4	4	5	5	6	7	7	7	8	8	9	9	9	10	10	10	11
120	2	3	3	4	4	5	5	6	7	7	8	8	8	9	9	9	10	10	10	11
110	2	3	4	4	4	5	5	6	7	7	8	8	8	9	9	9	10	10	10	11
100	2	3	4	4	4	5	6	6	7	7	8	8	8	9	9	9	10	10	10	11
95	2	3	4	4	4	5	6	6	7	7	8	8	8	9	9	9	10	10	10	
90	11	3	4	4	4	5	6	6	7	7	8	8	8	9	9	9	10	10		
85	11	11	11	11	10	10	10	7	7	7	8	8	9	9	9	9	10			
80	11	11	11	10	10	10	10	10	10	7	8	8	9	9	9	10				
75	11	11	10	10	10	10	10	10	10	9	8	8	9	9	9					
70	11	10	10	10	10	10	10	9	9	9	8	8	9	9						
65	11	10	10	10	10	9	9	9	9	9	9	9	9							
60	11	10	10	10	9	9	9	9	9	9	9	8								
55	11	10	10	9	9	9	9	9	9	8	8									
50	7	10	10	9	9	9	9	8	8	8										
45	7	8	10	9	9	9	8	8	8											
40	7	8	8	9	9	9	8	8												
35	7	8	8	9	9	9	8													
30	7	8	9	9	9	9														
25	7	8	9	9	9															
20	7	8	9	9																
15	7	8	9																	
10	7	8																		
5	7																			

The shorter segment is ahead followed by the longer one except where the wheel is overlined

Class M Engine

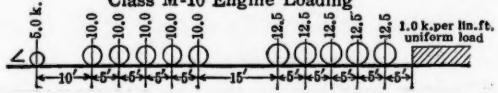


Papers.]

Note.—1 k. = 1 000 lb.

TABLE 18.—Maximum Moments, Shears and Reactions, Class M-10 Engine Loading, One Track.

Class M-10 Engine Loading



Span, in feet.	Maximum moment, in foot-kips.	Maximum shear, in kips.	Maximum floor-beam reaction, in kips.	Equivalent Uniform Load.			Span, in feet.	Maximum moment, in foot-kips.	Maximum shear, in kips.	Maximum floor-beam reaction, in kips.	Equivalent Uniform Load.		
				Moment.	Shear.	Reaction.					Moment.	Shear.	Reaction.
10	35.2	18.8	25.0	2.81	3.75	2.50	46	549.3	53.7	77.7	2.08	2.33	1.69
11	41.1	20.5	28.4	2.71	3.72	2.58	48	585.4	55.0	80.3	2.03	2.29	1.67
12	50.0	21.9	31.2	2.78	3.65	2.60	50	622.1	56.2	82.8	1.99	2.25	1.66
13	59.4	23.1	33.7	2.81	3.55	2.59	52	660.3	57.8	85.3	1.95	2.22	1.64
14	68.8	24.1	35.7	2.81	3.44	2.55	54	702.3	59.4	87.7	1.93	2.20	1.62
15	78.1	25.0	37.5	2.78	3.33	2.50	56	744.8	61.2	90.0	1.90	2.18	1.61
16	87.5	26.6	39.1	2.73	3.32	2.44	58	788.0	62.9	92.2	1.87	2.17	1.59
17	96.8	27.9	40.6	2.68	3.29	2.39	60	833.6	64.6	94.4	1.85	2.15	1.57
18	106.2	29.2	41.9	2.62	3.24	2.33	62	881.9	66.1	96.6	1.84	2.13	1.56
19	116.6	30.3	43.2	2.58	3.19	2.27	64	930.9	67.6	98.8	1.82	2.11	1.54
20	128.9	31.2	44.4	2.58	3.12	2.22	66	980.4	69.0	100.8	1.80	2.09	1.53
21	141.2	32.7	45.5	2.56	3.12	2.17	68	1 032.7	70.4	102.9	1.79	2.07	1.51
22	156.2	34.1	46.6	2.58	3.10	2.12	70	1 086.3	71.8	104.8	1.77	2.05	1.50
23	171.9	35.3	47.6	2.60	3.07	2.07	72	1 144.2	73.0	106.7	1.77	2.03	1.47
24	187.5	36.5	48.6	2.60	3.04	2.02	74	1 202.8	74.2	108.6	1.76	2.01	1.46
25	208.1	37.5	49.5	2.60	3.00	1.98	76	1 268.0	75.4	110.4	1.75	1.98	1.45
26	218.8	38.5	50.8	2.59	2.96	1.95	78	1 326.6	76.5	112.2	1.74	1.96	1.44
27	234.4	39.4	52.0	2.57	2.92	1.93	80	1 391.0	77.5	113.9	1.74	1.94	1.42
28	250.0	40.3	53.2	2.55	2.88	1.90	82	1 455.7	78.5	115.7	1.73	1.91	1.41
29	265.6	41.2	54.3	2.53	2.84	1.87	84	1 521.5	79.4	117.5	1.72	1.89	1.40
30	281.2	42.1	55.4	2.50	2.81	1.85	86	1 587.8	80.3	119.2	1.72	1.87	1.39
31	296.9	42.9	56.8	2.47	2.77	1.83	88	1 654.7	81.1	121.0	1.71	1.84	1.38
32	312.8	43.7	58.1	2.44	2.73	1.82	90	1 722.2	81.9	122.6	1.70	1.82	1.36
33	328.8	44.5	59.4	2.42	2.70	1.80	92	1 790.3	82.7	124.3	1.69	1.80	1.35
34	344.9	45.3	60.6	2.39	2.66	1.78	94	1 859.2	83.4	125.9	1.68	1.78	1.34
35	361.2	46.1	61.8	2.36	2.63	1.76	96	1 928.5	84.7	127.5	1.67	1.77	1.33
36	377.6	46.8	63.3	2.33	2.60	1.76	98	1 998.5	86.1	129.1	1.66	1.76	1.32
37	394.1	47.7	64.9	2.30	2.58	1.75	100	2 071.8	87.4	130.6	1.66	1.75	1.31
38	410.8	48.3	66.3	2.28	2.54	1.74	125	3 036.3	102.9	149.9	1.56	1.65	1.20
39	427.6	49.0	67.7	2.25	2.51	1.74	150	4 109.1	117.4	170.8	1.46	1.56	1.14
40	444.6	49.7	69.1	2.22	2.48	1.73	175	5 286.1	131.4	192.8	1.38	1.50	1.10
42	478.6	51.1	72.1	2.17	2.43	1.72	200	6 555.2	144.9	215.6	1.31	1.45	1.08
44	513.9	52.4	75.0	2.12	2.38	1.70	250	9 370.3	171.4	262.5	1.20	1.37	1.05

Note.—1 k. = 1 000 lb.

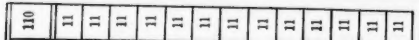
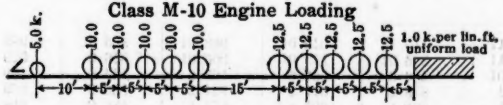


TABLE 19.—(*Continued.*)

Number of panels.	Panel point.	Panel Lengths.									
		13′-6″	14′-0″	14′-6″	15′-0″	15′-6″	16′-0″	16′-6″	17′-0″	17′-6″	18′-0″	18′-6″
8	⅛	404	427	451	475	501	527	554	581	608	636	664
4	¼	545	576	608	641	683	728	769	812	857	902	947
	¾	702	745	788	831	880	930	980	1 080	1 061	1 140	1 200
5	⅛	692	741	788	835	884	933	988	1 038	1 088	1 133	1 183
	⅖	906	1 065	1 135	1 205	1 269	1 359	1 437	1 516	1 595	1 675	1 756
6	⅛	829	880	930	981	1 033	1 084	1 137	1 191	1 245	1 306	1 370
	⅜	1 302	1 387	1 475	1 563	1 651	1 741	1 831	1 922	2 017	2 119	2 208
	⅝	1 414	1 515	1 616	1 719	1 822	1 927	2 033	2 143	2 255	2 368	2 482
7	⅐	932	988	1 042	1 098	1 164	1 232	1 300	1 370	1 441	1 514	1 587
	⅔	1 560	1 657	1 754	1 852	1 950	2 050	2 150	2 250	2 352	2 468	2 588
	⅞	1 853	1 973	2 097	2 223	2 350	2 479	2 608	2 739	2 871	3 005	3 155
8	⅛	1 033	1 100	1 170	1 242	1 316	1 392	1 469	1 548	1 628	1 709	1 793
	⅜	1 772	1 873	1 976	2 097	2 220	2 345	2 473	2 602	2 734	2 872	3 013
	⅝	2 223	2 362	2 503	2 645	2 789	2 936	3 099	3 264	3 436	3 611	3 788
	⅞	2 368	2 530	2 675	2 831	2 995	3 160	3 328	3 497	3 669	3 849	4 018
9	⅑	1 151	1 225	1 301	1 379	1 461	1 545	1 631	1 718	1 807	1 898	1 991
	⅓	1 961	2 113	2 247	2 388	2 533	2 665	2 810	2 957	3 108	3 261	3 419
	⅝	2 549	2 698	2 862	3 038	3 222	3 409	3 599	3 791	3 988	4 187	4 394
	⅞	2 860	3 038	3 225	3 415	3 608	3 804	4 008	4 215	4 425	4 638	4 853
10	1/10	1 263	1 345	1 428	1 514	1 604	1 696	1 789	1 885	1 983	2 083	2 185
	2/10	2 210	2 356	2 505	2 658	2 813	2 972	3 134	3 299	3 468	3 639	3 814
	3/10	2 864	3 044	3 241	3 441	3 650	3 863	4 080	4 300	4 524	4 752	4 983
	4/10	3 314	3 530	3 749	3 973	4 199	4 430	4 664	4 905	5 153	5 403	5 658
	5/10	3 455	3 669	3 886	4 106	4 330	4 556	4 792	5 031	5 273	5 519	5 767

TABLE 19.—MAXIMUM MOMENTS, IN FOOT-KIPS, FOR TRUSS BRIDGES, CLASS M-10 ENGINE-LOADING, ONE TRACK.

Class M-10 Engine Loading



PANEL LENGTHS.

Number of panels.	Panel point.	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"	10'-6"	11'-0"	11'-6"	12'-0"	12'-6"	13'-0"
3	1/3	167	188	208	229	250	271	292	314	336	358	380
4	1/4	236	262	287	314	341	368	396	425	454	484	514
	2/4	313	345	377	410	444	478	512	547	583	619	660
5	1/5	289	320	351	382	415	448	483	519	561	608	649
	2/5	429	472	516	561	609	654	702	751	809	870	932
6	1/6	335	370	406	447	490	537	584	631	679	729	779
	2/6	527	581	636	692	754	826	899	976	1 056	1 138	1 220
	3/6	583	639	692	766	831	905	980	1 056	1 140	1 230	1 320
7	1/7	377	427	475	523	571	622	673	723	775	826	880
	2/7	620	689	762	845	929	1 013	1 098	1 187	1 277	1 370	1 465
	3/7	732	810	895	985	1 077	1 179	1 290	1 394	1 504	1 618	1 734
8	1/8	436	485	536	588	639	691	747	801	855	910	967
	2/8	728	812	902	993	1 084	1 180	1 277	1 375	1 473	1 572	1 672
	3/8	888	993	1 100	1 217	1 336	1 456	1 578	1 706	1 835	1 965	2 097
	4/8	930	1 030	1 140	1 260	1 381	1 515	1 650	1 788	1 921	2 069	2 217
9	1/9	483	536	592	646	700	755	809	871	938	1 007	1 078
	2/9	838	930	1 027	1 125	1 224	1 323	1 424	1 525	1 627	1 729	1 853
	3/9	1 056	1 178	1 302	1 431	1 563	1 696	1 831	1 969	2 112	2 256	2 402
	4/9	1 137	1 277	1 418	1 562	1 708	1 862	2 018	2 182	2 348	2 517	2 687
10	1/10	530	584	639	694	749	817	886	957	1 030	1 106	1 183
	2/10	955	1 033	1 133	1 233	1 335	1 436	1 545	1 671	1 800	1 933	2 068
	3/10	1 207	1 341	1 476	1 618	1 761	1 858	2 053	2 202	2 351	2 503	2 674
	4/10	1 359	1 516	1 675	1 840	2 010	2 182	2 357	2 534	2 713	2 895	3 101
	5/10	1 381	1 548	1 719	1 892	2 069	2 255	2 444	2 636	2 831	3 036	3 244

TABLE 19.—(*Continued.*)

Number of panels.	Panel point.	Panel Lengths.									
		24'-6"	25'-0"	25'-6"	26'-0"	26'-6"	27'-0"	27'-6"	28'-0"	28'-6"	29'-0"	29'-6"
3	1/3	1 097	1 138	1 178	1 220	1 261	1 302	1 344	1 387	1 431	1 475	1 518
4	1/4	1 529	1 572	1 622	1 672	1 722	1 772	1 823	1 873	1 924	1 976	2 026
	2/4	1 904	2 069	2 143	2 217	2 292	2 368	2 444	2 520	2 597	2 675	2 753
5	1/5	1 866	1 933	2 000	2 068	2 136	2 210	2 282	2 356	2 430	2 505	2 581
	2/5	2 804	2 895	2 996	3 101	3 207	3 314	3 421	3 530	3 639	3 749	3 860
6	1/6	2 230	2 319	2 399	2 481	2 564	2 648	2 733	2 819	2 908	2 998	3 089
	2/6	3 535	3 668	3 792	3 922	4 054	4 187	4 325	4 464	4 604	4 746	4 889
	3/6	3 974	4 106	4 240	4 375	4 511	4 650	4 792	4 935	5 079	5 225	5 371
7	1/7	2 591	2 694	2 778	2 873	2 969	3 067	3 166	3 266	3 368	3 471	3 576
	2/7	4 288	4 398	4 549	4 707	4 867	5 029	5 192	5 357	5 526	5 698	5 872
	3/7	5 067	5 230	5 414	5 590	5 768	5 947	6 129	6 314	6 503	6 694	6 886
8	1/8	2 981	3 086	3 148	3 251	3 361	3 473	3 586	3 700	3 816	3 934	4 053
	2/8	4 916	5 097	5 280	5 465	5 653	5 842	6 034	6 229	6 425	6 624	6 827
	3/8	6 111	6 328	6 538	6 755	6 974	7 195	7 419	7 645	7 875	8 111	8 349
	4/8	6 925	6 581	6 745	6 960	7 178	7 397	7 619	7 842	8 068	8 296	8 525
9	1/9	3 261	3 379	3 499	3 621	3 744	3 869	3 996	4 125	4 255	4 387	4 520
	2/9	5 577	5 783	5 993	6 205	6 420	6 637	6 858	7 081	7 306	7 535	7 766
	3/9	7 124	7 375	7 630	7 886	8 146	8 409	8 675	8 944	9 216	9 494	9 777
	4/9	7 696	7 953	8 213	8 483	8 754	9 028	9 304	9 583	9 865	10 150	10 438
10	1/10	3 596	3 716	3 849	3 984	4 120	4 259	4 400	4 542	4 687	4 834	4 983
	2/10	6 225	6 458	6 693	6 932	7 174	7 419	7 668	7 919	8 174	8 432	8 693
	3/10	8 114	8 404	8 697	8 994	9 294	9 599	9 907	10 218	10 533	10 852	11 180
	4/10	9 033	9 340	9 652	9 972	10 296	10 624	10 955	11 290	11 628	11 970	12 312
	5/10	9 061	9 370	9 686	10 008	10 336	10 670	11 011	11 358	11 711	12 070	12 436

TABLE 19.—(*Continued.*)

Number of panels.	Panel point.	Panel Lengths.										
		19'-0"	19'-6"	20'-0"	20'-6"	21'-0"	21'-6"	22'-0"	22'-6"	23'-0"	23'-6"	24'-0"
3	1/6	602	721	754	791	826	863	899	935	976	1 016	1 056
	1/4	903	1 039	1 084	1 132	1 180	1 229	1 277	1 326	1 375	1 424	1 473
4	1/4	1 260	1 330	1 381	1 448	1 515	1 583	1 650	1 719	1 788	1 857	1 927
	3/4	1 283	1 284	1 335	1 386	1 437	1 490	1 545	1 603	1 671	1 735	1 800
5	1/6	1 840	1 925	2 010	2 096	2 182	2 269	2 357	2 445	2 534	2 623	2 713
	1/6	1 495	1 502	1 569	1 639	1 710	1 783	1 856	1 931	2 006	2 083	2 160
6	2/6	2 305	2 402	2 500	2 599	2 698	2 804	2 920	3 038	3 160	3 284	3 409
	3/6	2 507	2 714	2 831	2 954	3 077	3 212	3 328	3 455	3 588	3 712	3 842
	1/7	1 663	1 739	1 816	1 897	1 979	2 063	2 148	2 234	2 321	2 410	2 500
7	2/7	2 711	2 835	2 961	3 088	3 218	3 356	3 496	3 638	3 786	3 935	4 086
	3/7	3 303	3 604	3 604	3 787	3 911	4 067	4 229	4 393	4 559	4 726	4 896
	1/8	1 877	1 964	2 052	2 148	2 286	2 381	2 427	2 525	2 624	2 725	2 827
	2/8	3 155	3 300	3 447	3 596	3 748	3 901	4 057	4 216	4 387	4 561	4 738
8	3/8	3 967	4 149	4 333	4 519	4 708	4 899	5 092	5 286	5 499	5 694	5 909
	4/8	4 195	4 375	4 556	4 745	4 935	5 128	5 322	5 519	5 717	5 918	6 120
	1/9	2 085	2 181	2 279	2 381	2 485	2 591	2 698	2 807	2 918	3 031	3 145
	2/9	3 581	3 746	3 914	4 084	4 258	4 434	4 612	4 794	4 978	5 171	5 373
9	3/9	4 604	4 817	5 033	5 252	5 474	5 699	5 927	6 158	6 392	6 631	6 878
	4/9	5 071	5 292	5 515	5 742	5 975	6 213	6 453	6 696	6 942	7 190	7 442
	1/10	2 290	2 396	2 504	2 614	2 728	2 844	2 963	3 083	3 206	3 330	3 457
	2/10	3 934	4 179	4 368	4 559	4 754	4 952	5 153	5 358	5 565	5 776	5 996
	3/10	5 222	5 466	5 714	5 965	6 220	6 478	6 741	7 007	7 276	7 549	7 828
10	4/10	5 916	6 177	6 443	6 711	6 984	7 265	7 551	7 840	8 133	8 429	8 729
	5/10	6 019	6 273	6 531	6 793	7 069	7 342	7 619	7 903	8 191	8 461	8 758

TABLE 19.—(*Continued.*)

Number of panels	Panel point.	Panel Lengths.										
		35'-6"	36'-0"	37'-0"	38'-0"	39'-0"	40'-0"	41'-0"	42'-0"	43'-0"	44'-0"	45'-0"
3	1/6	2 064	2 112	2 208	2 305	2 402	2 500	2 599	2 698	2 804	2 930	3 038
4	1/4	2 803	2 872	3 013	3 155	3 295	3 447	3 596	3 748	3 901	4 057	4 216
	3/4	3 555	3 642	4 018	4 195	4 370	4 556	4 745	4 985	5 128	5 322	5 519
5	1/6	3 553	3 639	3 814	3 994	4 179	4 368	4 559	4 754	4 952	5 153	5 358
	2/3	5 277	5 403	5 658	5 916	6 177	6 449	6 711	6 984	7 265	7 551	7 840
6	1/6	4 264	4 369	4 581	4 798	5 019	5 248	5 481	5 719	5 960	6 206	6 456
	2/3	6 712	6 876	7 207	7 544	7 886	8 233	8 586	8 944	9 317	9 692	10 069
	5/6	7 232	7 397	7 780	8 068	8 410	8 763	9 122	9 496	9 878	10 270	10 671
7	1/7	4 952	5 072	5 325	5 580	5 840	6 105	6 380	6 660	6 945	7 235	7 530
	2/7	8 101	8 298	8 705	9 120	9 541	9 970	10 405	10 848	11 298	11 757	12 220
	3/7	9 344	9 563	10 006	10 456	10 913	11 380	11 866	12 364	12 874	13 396	13 930
8	1/8	5 636	5 767	6 054	6 346	6 646	6 952	7 264	7 586	7 914	8 249	8 589
	2/8	9 460	9 694	10 171	10 662	11 168	11 672	12 190	12 717	13 254	13 799	14 359
	3/8	11 400	11 673	12 227	12 789	13 361	13 947	14 558	15 177	15 815	16 466	17 155
	4/8	11 640	11 926	12 510	13 110	13 726	14 358	15 006	15 670	16 350	17 046	17 758
9	1/9	6 280	6 449	6 773	7 104	7 442	7 788	8 139	8 508	8 874	9 252	9 638
	2/9	10 797	11 067	11 617	12 183	12 762	13 351	13 952	14 563	15 185	15 819	16 467
	3/9	13 490	13 747	14 410	15 085	15 772	16 477	17 206	17 958	18 718	19 501	20 302
	4/9	14 338	14 691	15 421	16 171	16 941	17 731	18 541	19 371	20 221	21 091	21 904
10	1/10	6 947	7 124	7 485	7 854	8 231	8 616	9 007	9 412	9 836	10 249	10 679
	2/10	12 119	12 425	13 049	13 689	14 345	15 015	15 696	16 393	17 101	17 821	18 558
	3/10	15 413	15 794	16 567	17 355	18 157	18 961	19 831	20 703	21 195	22 509	23 443
	4/10	16 992	17 421	18 297	19 197	20 121	21 060	22 041	23 057	24 057	25 101	26 169
	5/10	17 311	17 758	18 670	19 608	20 570	21 558	22 570	23 608	24 670	25 758	26 870

TABLE 19.—(*Continued*)

Number of panels.	Panel point.	Panel Lengths.										
		30′-0″	30′-6	31′-0″	31′-6″	32′-0″	32′-6″	33′-0″	33′-6″	34′-0″	34′-6″	35′-0″
3	1/3	1 563	1 607	1 651	1 696	1 741	1 785	1 831	1 876	1 922	1 969	2 017
4	1/4	2 097	2 158	2 220	2 282	2 345	2 409	2 473	2 537	2 602	2 668	2 734
	2/4	2 831	2 913	2 995	3 077	3 160	3 244	3 328	3 412	3 497	3 583	3 669
5	1/5	2 658	2 735	2 813	2 892	2 972	3 053	3 134	3 216	3 299	3 383	3 468
	2/5	3 973	4 085	4 199	4 314	4 430	4 546	4 664	4 783	4 905	5 028	5 153
6	1/6	3 181	3 274	3 369	3 464	3 560	3 659	3 756	3 856	3 956	4 058	4 160
	2/6	5 033	5 179	5 326	5 474	5 624	5 775	5 927	6 081	6 236	6 392	6 550
	3/6	5 519	5 667	5 817	5 968	6 120	6 273	6 428	6 584	6 745	6 906	7 069
7	1/7	3 684	3 793	3 903	4 014	4 127	4 241	4 356	4 473	4 591	4 710	4 830
	2/7	6 048	6 226	6 405	6 587	6 770	6 954	7 141	7 330	7 520	7 712	7 905
	3/7	7 080	7 276	7 474	7 673	7 874	8 077	8 281	8 487	8 697	8 911	9 127
8	1/8	4 173	4 298	4 424	4 551	4 680	4 811	4 943	5 076	5 211	5 348	5 486
	2/8	7 084	7 244	7 455	7 669	7 885	8 103	8 324	8 546	8 771	8 999	9 228
	3/8	8 589	8 832	9 077	9 324	9 573	9 825	10 080	10 336	10 595	10 861	11 130
	4/8	8 758	9 000	9 246	9 496	9 750	10 008	10 270	10 536	10 806	11 080	11 358
9	1/9	4 657	4 794	4 936	5 079	5 224	5 371	5 520	5 669	5 822	5 976	6 132
	2/9	8 001	8 242	8 485	8 731	8 980	9 231	9 485	9 742	10 002	10 264	10 529
	3/9	10 062	10 351	10 644	10 939	11 237	11 538	11 842	12 149	12 457	12 775	13 130
	4/9	10 731	11 033	11 341	11 653	11 971	12 293	12 621	12 963	13 291	13 633	14 006
10	1/10	5 134	5 287	5 442	5 600	5 761	5 925	6 090	6 257	6 427	6 598	6 771
	2/10	8 958	9 227	9 501	9 779	10 061	10 345	10 633	10 923	11 217	11 515	11 815
	3/10	11 511	11 847	12 185	12 528	12 874	13 224	13 578	13 935	14 296	14 663	15 036
	4/10	12 669	13 092	13 401	13 776	14 157	14 544	14 937	15 336	15 741	16 152	16 569
	5/10	12 808	13 186	13 570	13 961	14 358	14 761	15 170	15 586	16 008	16 436	16 870

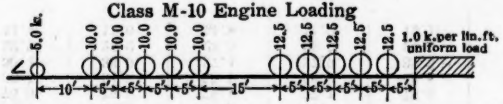
TABLE 20.—(*Continued*).

PANEL LENGTHS.

Number of panels.	Panel point.	13′-6″	14′-0″	14′-6″	15′-0″	15′-6″	16′-0″	16′-6″	17′-0″	17′-6″	18′-0″	18′-6″
3	1/3	30.0	30.6	31.1	31.7	32.4	33.0	33.6	34.2	34.8	35.4	35.9
	2/3	8.9	9.2	9.5	9.7	10.1	10.4	10.7	11.0	11.3	11.6	11.8
4	1/4	40.4	41.2	42.1	43.3	44.4	45.5	46.5	47.8	49.0	50.1	51.2
	2/4	20.8	21.2	21.5	21.8	22.2	22.7	23.2	23.6	24.0	24.5	25.0
	3/4	5.9	6.0	6.1	6.3	6.5	6.8	7.1	7.3	7.6	7.8	8.0
5	1/5	51.4	53.0	54.5	55.7	57.0	58.3	59.5	60.8	61.9	63.0	64.0
	2/5	30.9	31.7	32.5	33.4	34.3	35.1	35.9	36.7	37.5	38.4	39.2
	3/5	15.7	16.1	16.4	16.7	17.1	17.4	17.8	18.1	18.4	18.8	19.2
6	1/6	61.4	62.8	64.1	65.4	66.7	67.9	69.0	70.1	71.2	72.5	74.0
	2/6	41.8	43.0	44.2	45.3	46.3	47.3	48.3	49.3	50.2	51.0	51.8
	3/6	24.9	25.6	26.3	26.9	27.7	28.5	29.2	29.9	30.6	31.2	31.9
	4/6	13.1	13.4	13.7	13.9	14.1	14.3	14.5	14.7	14.9	15.1	15.4
7	1/7	69.1	70.5	71.9	73.2	75.1	77.0	78.8	80.6	82.4	84.1	85.7
	2/7	51.7	52.9	54.2	55.4	56.6	57.7	58.9	60.0	61.0	62.2	63.3
	3/7	34.9	35.9	36.9	38.9	39.0	40.0	40.8	41.6	42.3	43.0	43.7
	4/7	21.1	21.6	22.0	22.5	23.1	23.7	24.3	24.9	25.5	26.1	26.7
	5/7	11.2	11.5	11.7	11.9	12.1	12.3	12.5	12.6	12.8	13.0	13.2
8	1/8	76.4	78.5	80.7	82.8	84.9	87.0	89.0	91.0	93.0	94.9	96.8
	2/8	59.6	61.0	62.4	63.9	65.3	66.7	68.1	69.6	71.0	72.5	74.0
	3/8	44.1	45.3	46.5	47.4	48.3	49.3	50.2	51.1	52.0	53.1	54.1
	4/8	30.1	31.1	32.0	32.8	33.6	34.4	35.2	35.9	36.5	37.1	37.7
	5/8	18.2	18.6	19.1	19.5	20.0	20.4	20.9	21.5	22.1	22.7	23.2
	6/8	9.8	10.0	10.2	10.4	10.6	10.7	10.9	11.0	11.2	11.4	11.6
9	1/9	85.2	87.5	89.7	91.9	94.2	96.5	98.9	101.0	103.2	105.4	107.7
	2/9	68.0	69.7	71.4	73.1	74.8	76.5	78.2	80.0	81.8	83.5	85.8
	3/9	52.8	53.9	55.1	56.2	57.4	58.5	59.7	60.8	62.2	63.5	64.9
	4/9	39.3	40.3	41.3	42.1	42.9	43.7	44.4	45.1	45.8	46.7	47.6
	5/9	26.4	27.2	28.0	28.7	29.4	30.2	30.9	31.5	32.2	32.8	33.4
	6/9	16.2	16.6	17.0	17.4	17.8	18.2	18.6	19.0	19.4	19.8	20.3
	7/9	8.7	8.9	9.1	9.3	9.4	9.5	9.7	9.8	9.9	10.1	10.2
10	1/10	93.5	96.0	98.4	100.9	103.5	106.0	108.4	110.9	113.3	115.8	118.2
	2/10	76.0	77.9	79.9	81.9	84.0	86.0	88.0	90.1	92.1	94.2	96.2
	3/10	60.5	61.9	63.4	64.9	66.5	68.1	69.6	71.2	72.8	74.4	75.9
	4/10	46.9	48.0	49.1	50.3	51.4	52.5	53.6	54.8	55.9	57.0	58.1
	5/10	34.1	35.0	35.9	36.8	37.7	38.6	39.5	40.4	41.1	41.8	42.5
	6/10	23.4	23.9	24.5	25.1	25.7	26.2	26.8	27.4	27.9	28.5	29.1
	7/10	14.5	14.9	15.3	15.6	16.0	16.4	16.7	17.1	17.5	17.9	18.2
	8/10	7.9	8.0	8.2	8.3	8.5	8.6	8.7	8.8	8.9	9.1	9.2

TABLE 20.—Maximum Shears, in Kips, for Truss Bridges, Class M-10 Engine-Loading, One Track.

Class M-10 Engine Loading



Number of panels.	Panel point.	Panel Lengths. 8′-0″	8′-6″	9′-0″	9′-6″	10′-0″	10′-6″	11′-0″	11′-6″	12′-0″	12′-6″	13′-0″
3	1/8	20.8	22.0	23.1	24.1	25.0	25.8	26.6	27.3	28.0	28.6	29.3
	2/8	5.7	5.9	6.0	6.1	6.2	6.7	7.2	7.6	8.0	8.3	8.6
4	1/4	29.5	30.7	31.9	33.0	34.1	35.1	36.0	36.9	37.8	38.7	39.6
	2/4	13.7	14.7	15.6	16.4	17.2	17.9	18.5	19.0	19.5	20.0	20.4
	3/4	4.3	4.4	4.5	4.6	4.7	4.9	5.1	5.3	5.5	5.6	5.8
5	1/5	36.2	37.6	38.9	40.2	41.5	42.7	43.9	45.1	46.7	48.3	49.9
	2/5	21.9	22.7	23.5	24.3	25.1	25.9	26.7	27.5	28.3	29.1	30.0
	3/5	10.6	11.2	11.7	12.1	12.5	13.1	13.6	14.1	14.6	15.0	15.4
6	1/6	41.9	43.6	45.1	47.0	49.0	51.0	53.1	54.9	56.6	58.2	59.8
	2/6	28.6	29.4	30.3	31.5	32.7	34.0	35.3	36.6	37.9	39.2	40.5
	3/6	18.2	19.0	19.7	20.3	20.8	21.3	21.8	22.2	22.7	23.4	24.1
	4/6	8.9	9.3	9.7	10.1	10.4	10.9	11.4	11.8	12.1	12.5	12.8
7	1/7	47.2	50.2	52.8	55.0	57.1	59.1	61.1	62.9	64.6	66.1	67.7
	2/7	34.4	35.8	37.4	39.3	41.1	42.9	44.6	46.1	47.6	49.0	50.4
	3/7	24.5	25.4	26.3	27.2	28.2	29.1	30.0	30.9	31.8	32.8	33.8
	4/7	15.6	16.3	16.9	17.4	17.9	18.4	18.8	19.3	19.7	20.2	20.7
	5/7	7.6	8.0	8.3	8.6	8.9	9.3	9.7	10.1	10.4	10.7	11.0
8	1/8	54.5	57.0	59.5	61.8	63.8	65.8	67.8	69.6	71.3	72.8	74.3
	2/8	41.1	43.0	45.0	46.9	48.8	50.8	52.7	54.1	55.5	56.9	58.2
	3/8	30.1	31.5	32.8	34.2	35.6	36.9	38.3	39.5	40.6	41.8	42.9
	4/8	21.5	22.1	22.7	23.4	24.1	24.9	25.6	26.4	27.2	28.1	29.1
	5/8	13.7	14.2	14.7	15.2	15.6	16.0	16.3	16.6	17.0	17.4	17.8
	6/8	6.6	7.0	7.3	7.6	7.8	8.2	8.5	8.8	9.1	9.4	9.6
9	1/9	60.3	62.9	65.7	67.9	70.0	71.8	73.6	75.6	78.1	80.5	82.9
	2/9	47.8	50.1	52.5	54.5	56.4	58.0	59.6	61.2	62.9	64.6	66.3
	3/9	36.4	37.9	39.6	41.4	43.1	44.8	46.4	47.9	49.3	50.5	51.6
	4/9	26.7	27.8	28.9	30.0	31.1	32.3	33.5	34.7	35.9	37.0	38.1
	5/9	19.1	19.6	20.2	20.8	21.4	22.1	22.8	23.5	24.2	24.9	25.7
	6/9	12.1	12.6	13.1	13.5	13.9	14.2	14.5	14.8	15.1	15.5	15.8
	7/9	5.9	6.2	6.5	6.7	7.0	7.3	7.6	7.9	8.1	8.3	8.5
10	1/10	66.2	68.6	71.0	73.0	74.9	77.7	80.5	83.2	85.9	88.5	91.0
	2/10	54.3	56.6	59.0	60.7	62.5	64.2	66.2	68.1	70.1	72.0	74.0
	3/10	42.8	44.5	46.1	47.8	49.4	51.1	52.7	54.4	56.0	57.5	59.0
	4/10	32.7	34.2	35.7	37.2	38.7	40.1	41.4	42.5	43.6	44.7	45.8
	5/10	24.0	25.0	26.0	26.9	27.8	28.7	29.6	30.5	31.4	32.3	33.2
	6/10	17.2	17.7	18.2	18.7	19.3	19.9	20.5	21.1	21.6	22.2	22.8
	7/10	10.9	11.4	11.8	12.2	12.5	12.8	13.1	13.3	13.6	13.9	14.2
	8/10	5.3	5.6	5.8	6.1	6.3	6.6	6.8	7.1	7.2	7.5	7.7

TABLE 20.—(*Continued*).

TABLE 20.—(*Continued.*)

PANEL LENGTHS.

Number of panels.	Panel point.	24′-6″	25′-0″	25′-6″	26′-0″	26′-6″	27′-0″	27′-6″	28′-0″	28′-6″	29′-0″	29′-6″
3	1/3	44.7	45.5	46.2	46.9	47.6	48.3	48.9	49.5	50.2	50.9	51.5
	2/3	14.1	14.3	14.4	14.5	14.7	14.8	14.9	15.1	15.2	15.3	15.5
4	1/4	62.2	62.9	63.6	64.3	65.0	65.6	66.2	66.9	67.5	68.1	69.0
	2/4	30.8	31.3	31.8	32.3	32.8	33.4	33.9	34.4	34.8	35.2	35.7
	3/4	9.9	10.0	10.1	10.2	10.3	10.4	10.5	10.6	10.7	10.8	10.8
5	1/5	76.1	77.3	78.4	79.6	80.7	81.9	83.0	84.1	85.3	86.4	87.5
	2/5	47.8	48.4	48.9	49.5	50.0	50.4	50.9	51.4	52.0	52.5	53.0
	3/5	23.7	24.1	24.5	24.9	25.3	25.7	26.1	26.5	26.8	27.2	27.5
6	1/6	91.4	92.8	94.2	95.5	96.7	98.0	99.3	100.5	102.0	103.4	104.8
	2/6	61.7	62.7	63.7	64.7	65.6	66.5	67.5	68.4	69.3	70.2	71.1
	3/6	38.8	39.2	39.6	40.1	40.6	41.0	41.4	41.9	42.3	42.7	43.1
	4/6	19.3	19.6	19.9	20.3	20.6	20.9	21.2	21.5	21.8	22.1	22.4
7	1/7	105.7	107.3	108.9	110.4	112.0	113.7	115.2	116.7	118.2	119.7	121.2
	2/7	76.8	78.0	79.2	80.4	81.6	82.7	83.8	85.0	86.2	87.3	88.4
	3/7	52.1	52.9	53.7	54.5	55.3	56.1	56.8	57.6	58.4	59.1	59.8
	4/7	32.7	33.0	33.4	33.8	34.2	34.5	34.9	35.3	35.6	36.0	36.3
	5/7	16.2	16.4	16.7	17.0	17.3	17.6	18.0	18.4	18.9	19.3	19.7
8	1/8	119.5	121.4	123.2	125.0	126.8	128.6	130.4	132.2	133.9	135.6	137.3
	2/8	91.4	92.8	94.1	95.5	96.9	98.3	99.6	101.0	102.3	103.6	104.9
	3/8	66.5	67.5	68.4	69.4	70.3	71.3	72.2	73.2	74.2	75.2	76.2
	4/8	44.9	45.6	46.3	47.0	47.7	48.4	49.1	49.8	50.4	51.0	51.6
	5/8	28.2	28.5	28.9	29.2	29.5	29.8	30.2	30.5	30.8	31.2	31.5
	6/8	13.9	14.1	14.4	14.6	14.9	15.1	15.4	15.6	15.8	16.0	16.2
9	1/9	133.1	135.2	137.2	139.2	141.2	143.2	145.3	147.3	149.3	151.2	153.2
	2/9	105.2	106.8	108.5	110.1	111.7	113.3	114.9	116.4	118.0	119.6	121.2
	3/9	80.3	81.5	82.7	83.8	85.0	86.3	87.5	88.7	89.9	91.2	92.4
	4/9	58.4	59.2	60.1	61.0	61.9	62.8	63.7	64.6	65.4	66.2	69.1
	5/9	39.7	40.3	40.9	41.4	42.0	42.6	43.2	43.7	44.3	44.9	45.5
	6/9	24.9	25.2	25.5	25.8	26.1	26.4	26.7	27.0	27.3	27.6	27.9
	7/9	12.3	12.5	12.6	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.2
10	1/10	146.3	148.6	151.0	153.3	155.5	157.8	160.1	162.2	164.4	166.6	168.9
	2/10	118.8	120.6	122.5	124.4	126.2	128.0	129.8	131.6	133.4	135.2	137.0
	3/10	93.9	95.3	96.7	98.1	99.5	101.0	102.5	103.9	105.3	106.7	108.2
	4/10	71.7	72.8	73.9	75.0	76.1	77.1	78.2	79.3	80.3	81.3	82.4
	5/10	51.8	52.7	53.5	54.3	55.1	56.0	56.8	57.6	58.4	59.2	60.0
	6/10	35.5	35.9	36.4	36.9	37.4	37.9	38.3	38.8	39.3	39.8	40.4
	7/10	22.4	22.7	22.9	23.2	23.5	23.7	24.0	24.2	24.4	24.6	24.8
	8/10	11.1	11.3	11.4	11.6	11.8	11.9	12.1	12.3	12.5	12.6	12.8

TABLE 20.—(*Continued.*)

PANEL LENGTHS.

Number of panels.	Panel point.	19′-0″	19′-6″	20′-6″	20′-6″	21′-0″	21′-6″	22′-0″	22′-6″	23′-0″	23′-6″	24′-0″
3	1/3	36.4	37.0	37.7	38.5	39.3	40.0	40.8	41.5	42.4	43.2	44.0
	2/3	12.1	12.3	12.5	12.7	12.9	13.1	13.3	13.4	13.6	13.8	14.0
4	1/4	52.2	53.3	54.2	55.2	56.2	57.1	58.0	58.9	59.8	60.6	61.4
	2/4	25.4	25.8	26.3	26.8	27.3	27.8	28.3	28.8	29.3	29.8	30.3
	3/4	8.2	8.4	8.6	8.8	8.9	9.1	9.2	9.4	9.5	9.6	9.8
5	1/5	65.0	65.9	66.8	67.6	68.4	69.3	70.2	71.4	72.7	73.8	75.0
	2/5	40.1	40.9	41.8	42.5	43.2	43.9	44.7	45.4	46.0	46.6	47.2
	3/5	19.6	19.9	20.3	20.7	21.0	21.4	21.8	22.1	22.6	23.0	23.4
6	1/6	75.5	77.0	78.5	80.0	81.4	82.9	84.3	85.8	87.2	88.5	90.0
	2/6	52.6	53.4	54.2	54.9	55.7	56.5	57.3	58.0	59.0	59.9	60.8
	3/6	32.6	33.3	33.9	34.5	35.1	35.7	36.3	36.9	37.4	37.8	38.3
	4/6	15.8	16.1	16.3	16.7	17.0	17.3	17.7	18.0	18.3	18.6	19.0
7	1/7	87.4	89.0	90.7	92.5	94.2	96.0	97.7	99.3	100.9	102.5	104.1
	2/7	64.4	65.5	66.5	67.5	68.6	69.6	70.6	71.9	73.2	74.4	75.6
	3/7	44.4	45.1	45.8	46.4	47.0	47.6	48.2	48.6	49.6	50.4	51.3
	4/7	27.3	27.9	28.6	29.1	29.6	30.1	30.6	31.1	31.5	31.9	32.3
	5/7	13.4	13.7	13.9	14.1	14.3	14.6	14.8	15.0	15.3	15.6	15.9
8	1/8	98.7	100.7	102.6	104.5	106.3	108.2	110.2	112.2	114.1	116.0	117.8
	2/8	75.5	76.9	78.4	79.9	81.4	82.8	84.2	85.7	87.1	88.5	90.0
	3/8	55.1	56.2	57.2	58.2	59.2	60.2	61.3	62.3	63.4	64.4	65.5
	4/8	38.3	38.9	39.5	40.1	40.7	41.3	41.9	42.5	43.1	43.7	44.3
	5/8	23.7	24.2	24.6	25.0	25.5	26.0	26.4	26.8	27.2	27.6	28.0
	6/8	11.8	12.0	12.2	12.4	12.5	12.7	12.9	13.1	13.3	13.5	13.7
9	1/9	109.9	112.0	114.0	116.2	118.2	120.4	122.7	124.8	126.8	128.9	131.1
	2/9	86.9	88.6	90.3	92.0	93.7	95.3	96.9	98.6	100.2	101.8	103.5
	3/9	66.3	67.6	69.0	70.2	71.5	72.8	74.0	75.3	76.6	77.9	79.1
	4/9	48.5	49.4	50.3	51.2	52.2	53.1	54.0	54.9	55.8	56.7	57.6
	5/9	34.0	34.5	35.0	35.5	35.9	36.4	36.8	37.4	38.0	38.6	39.1
	6/9	20.7	21.1	21.5	21.9	22.3	22.7	23.1	23 5	23.9	24.3	24.6
	7/9	10.4	10.6	10.7	10.9	11.1	11.2	11.4	11.6	11.7	11.9	12.1
10	1/10	120.6	122.9	125.2	127.5	130.0	132.3	134.7	137.0	139.4	141.8	144.1
	2/10	98.1	100.0	101.8	103.7	105.7	107.6	109.4	111.3	113.1	115.0	116.8
	3/10	77.5	79.1	80.7	82.2	83.7	85.2	86 7	88.2	89.6	91.1	92.5
	4/10	59.3	60.4	61.5	62.7	63.8	64.9	66.1	67.2	68.3	69.5	70.7
	5/10	43.2	43.8	44.5	45.2	46.0	46.8	47.7	48.5	49.3	50.2	51.0
	6/10	29.7	30.2	30.8	31.4	31.9	32.5	33.1	33.6	34.1	34.6	35.0
	7/10	18.6	19.0	19.3	19.6	20.0	20.4	20.7	21.1	21.4	21.7	22.1
	8/10	9.4	9.5	9.6	9.8	10.0	10.1	10.3	10.4	10.6	10.8	10.9

TABLE 20.—(*Continued.*)

TABLE 20.—(*Continued.*)

PANEL LENGTHS.

Number of panels.	Panel point.	35′-6″	36′-0″	37′-0″	38′-0″	39′-0″	40′-0″	41′-0″	42′-0″	43′-0″	44′-0″	45′-0″
3	1/3	58.1	58.6	59.7	60.7	61.6	62.5	63.4	64.3	65.2	66.3	67.5
	2/3	17.2	17.4	17.8	18.1	18.5	18.9	19.3	19.7	20.0	20.4	20.8
4	1/4	79.0	79.8	81.4	83.0	84.6	86.2	87.8	89.3	90.8	92.3	93.7
	2/4	40.5	40.9	41.5	42.1	42.8	43.4	44.0	44.6	45.2	45.8	46.6
	3/4	12.1	12.2	12.5	12.7	12.9	13.2	13.4	13.6	13.9	14.2	14.4
5	1/5	100.1	101.1	103.0	105.0	107.1	109.2	111.2	113.2	115.2	117.1	119.1
	2/5	60.6	61.3	62.6	63.9	65.2	66.4	67.6	68.8	70.0	71.2	72.4
	3/5	31.2	31.5	32.0	32.5	33.0	33.4	33.8	34.2	34.6	35.1	35.7
6	1/6	120.1	121.2	123.7	126.2	128.7	131.2	133.7	136.2	138.7	141.2	143.6
	2/6	81.4	82.3	84.0	85.6	87.3	89.0	90.7	92.3	93.9	95.5	97.1
	3/6	49.3	49.9	51.0	52.0	53.0	54.0	54.9	55.9	56.9	57.9	58.8
	4/6	25.3	25.5	25.9	26.3	26.7	27.1	27.5	27.9	28.3	28.7	29.0
7	1/7	139.5	141.0	144.0	147.0	150.0	153.0	156.0	158.9	161.7	164.5	167.3
	2/7	101.3	102.3	104.5	106.6	108.7	110.8	112.9	115.0	117.0	119.1	121.1
	3/7	68.7	69.4	70.8	72.2	73.6	75.0	76.4	77.7	79.0	80.4	81.8
	4/7	41.6	42.0	42.9	43.8	44.6	45.4	46.2	47.1	47.9	48.8	49.6
	5/7	21.4	21.5	21.8	22.2	22.5	22.9	23.2	23.5	23.8	24.1	24.4
8	1/8	158.5	160.2	163.6	167.0	170.4	173.8	177.2	180.7	184.1	187.6	191.0
	2/8	120.5	121.8	124.5	127.1	129.7	132.2	134.7	137.3	139.8	142.3	144.7
	3/8	87.7	88.6	90.4	92.2	94.0	95.8	97.6	99.4	101.2	103.0	104.7
	4/8	59.2	59.9	61.0	62.2	63.5	64.8	66.1	67.3	68.4	69.6	70.8
	5/8	36.0	36.3	37.1	37.8	38.6	39.3	40.0	40.7	41.4	42.1	42.8
	6/8	18.5	18.7	18.9	19.2	19.5	19.8	20.1	20.4	20.7	20.9	21.2
9	1/9	177.1	179.0	183.0	187.0	190.9	194.7	198.2	202.4	206.4	210.3	214.2
	2/9	139.5	141.0	144.1	147.1	150.2	153.2	156.2	159.3	162.3	165.2	168.0
	3/9	106.3	107.5	109.7	111.8	114.1	116.4	118.6	120.9	123.1	125.4	127.7
	4/9	77.1	77.9	79.6	81.2	82.8	84.4	86.0	87.6	89.2	90.8	92.4
	5/9	52.2	52.7	53.8	54.8	55.9	57.0	58.1	59.2	60.2	61.3	62.3
	6/9	31.5	31.8	32.5	33.2	33.9	34.5	35.1	35.8	36.4	37.0	37.7
	7/9	16.4	16.5	16.8	17.0	17.3	17.5	17.8	18.0	18.2	18.4	18.6
10	1/10	195.7	198.0	202.3	206.7	211.1	215.4	219.7	224.1	228.5	232.9	237.3
	2/10	158.2	160.0	163.5	167.0	170.5	174.0	177.5	181.0	184.4	187.8	191.3
	3/10	124.8	126.2	129.0	131.6	134.3	136.9	139.6	142.3	145.1	147.7	150.3
	4/10	95.0	96.0	98.0	100.1	102.1	104.1	106.1	108.1	110.1	112.0	114.1
	5/10	68.8	69.5	70.9	72.4	73.9	75.4	76.9	78.3	79.7	81.1	82.5
	6/10	46.5	47.1	48.1	49.0	50.0	51.0	51.9	52.8	53.8	54.7	55.6
	7/10	28.0	28.3	28.9	29.5	30.1	30.7	31.3	31.6	32.5	33.0	33.6
	8/10	14.6	14.7	14.9	15.1	15.4	15.6	15.8	16.0	16.2	16.4	16.6

TABLE 20.—(*Continued*).

Number of panels.	Panel point.	Panel Lengths. 30′-0″	30′-6″	31′-0″	31′-6″	32′-0″	32′-6″	33′-0″	33′-6″	34′-0″	34′-6″	35′-0″
3	1/3	52.1	52.7	53.3	53.8	54.4	54.9	55.5	56.0	56.6	57.1	57.6
	2/3	15.6	15.8	15.9	16.0	16.2	16.4	16.5	16.6	16.8	17.0	17.1
4	1/4	69.9	70.7	71.5	72.4	73.3	74.1	74.9	75.7	76.5	77.3	78.1
	2/4	36.2	36.6	37.0	37.5	37.9	38.3	38.7	39.1	39.5	39.9	40.2
	3/4	10.9	11.0	11.1	11.2	11.4	11.5	11.6	11.7	11.8	11.9	12.0
5	1/5	88.6	89.6	90.7	91.8	92.8	93.8	94.8	95.9	97.0	98.0	99.0
	2/5	53.6	54.3	54.9	55.5	56.2	56.9	57.5	58.1	58.8	59.4	60.0
	3/5	27.8	28.2	28.5	28.8	29.2	29.5	29.8	30.1	30.4	30.7	31.0
6	1/6	106.0	107.3	108.7	110.0	111.2	112.3	113.6	115.0	116.2	117.6	119.0
	2/6	72.0	72.8	73.6	74.5	75.5	76.4	77.2	78.0	78.8	79.6	80.5
	3/6	43.5	44.0	44.6	45.1	45.7	46.2	46.7	47.2	47.8	48.3	48.8
	4/6	22.6	22.9	23.2	23.5	23.8	24.0	24.3	24.5	24.7	24.9	25.1
7	1/7	122.8	124.3	126.8	127.2	128.6	130.3	132.0	133.7	135.1	136.6	138.0
	2/7	89.4	90.5	91.6	92.7	93.8	94.9	95.9	97.0	98.0	99.1	100.2
	3/7	60.5	61.3	62.1	62.9	63.5	64.2	65.0	65.8	66.5	67.3	68.0
	4/7	36.6	37.1	37.6	38.0	38.5	39.0	39.4	39.8	40.3	40.8	41.2
	5/7	19.1	19.4	19.6	19.8	20.0	20.2	20.4	20.6	20.8	21.0	21.2
8	1/8	139.1	141.0	142.9	144.6	146.3	148.0	149.7	151.5	153.2	155.0	156.8
	2/8	106.3	107.6	108.9	110.2	111.5	112.9	114.2	115.4	116.7	118.0	119.2
	3/8	77.2	78.2	79.1	80.1	81.0	82.0	83.0	83.9	84.8	85.7	86.7
	4/8	52.2	52.8	53.4	54.0	54.6	55.3	56.0	56.7	57.3	58.0	58.6
	5/8	31.8	32.1	32.5	32.8	33.2	33.6	34.0	34.4	34.8	35.2	35.6
	6/8	16.4	16.6	16.8	17.0	17.2	17.4	17.6	17.8	18.0	18.1	18.3
9	1/9	155.2	157.2	159.2	161.2	163.2	165.2	167.3	169.4	171.3	173.2	175.2
	2/9	122.7	124.2	125.7	127.3	128.8	130.3	131.8	133.3	134.8	136.4	137.9
	3/9	93.5	94.7	95.9	97.1	98.3	99.4	100.6	101.7	102.8	104.0	105.2
	4/9	67.9	68.8	69.6	70.4	71.2	72.1	72.9	73.8	74.6	75.4	76.2
	5/9	46.0	46.5	47.1	47.7	48.2	48.8	49.4	49.9	50.5	51.0	51.6
	6/9	28.2	28.5	28.7	29.0	29.3	29.6	29.9	30.2	30.4	30.8	31.1
	7/9	14.4	14.6	14.7	14.9	15.1	15.3	15.5	15.7	15.8	16.0	16.2
10	1/10	171.1	173.3	175.5	177.8	180.0	182.2	184.5	186.8	189.0	191.3	193.5
	2/10	138.8	140.5	142.3	144.1	145.9	147.7	149.4	151.2	153.0	154.7	156.4
	3/10	109.6	111.0	112.4	113.8	115.2	116.6	118.0	119.4	120.7	122.0	123.4
	4/10	83.4	84.4	85.5	86.5	87.6	88.7	89.8	90.8	91.9	93.0	94.0
	5/10	60.7	61.5	62.2	62.9	63.6	64.4	65.2	65.9	66.6	67.4	68.1
	6/10	40.9	41.4	42.0	42.5	43.0	43.5	44.0	44.5	45.1	45.6	46.1
	7/10	25.0	25.3	25.6	25.8	26.1	26.4	26.6	26.9	27.2	27.5	27.7
	8/10	13.0	13.1	13.3	13.4	13.6	13.7	13.9	14.0	14.1	14.3	14.4

TABLE 20.—(*Continued.*)

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

THE WATER POWER PROBLEM
A SYMPOSIUM
Discussion*

BY MESSRS. J. B. LIPPINCOTT, C. S. JARVIS, D. C. HENNY, F. H. TIBBETTS, LOUIS C. HILL, C. H. BIRDSEYE, AND EDWARD H. SARGENT.

J. B. LIPPINCOTT,† M. AM. SOC. C. E.—The Colorado River is the great undeveloped water resource of the Southwest, and its greatest importance is the irrigation of the 2 000 000 acres of arid land below the Grand Canyon. The basic feature governing the liberation of the water of the Colorado from these reservoirs, should be for the benefit, primarily, of irrigation. Second in importance is the prevention of flood overflow of the ribbon-like valleys below the Grand Canyon, and the delta, including about 800 000 acres in the Imperial Valley. There is also an equally great area of delta land on the Colorado River, in Mexico, that is being rapidly developed, and protection of these lands from flood destruction is essential.

The entire late summer flow of the Colorado River near Yuma, Ariz., is now diverted, and the time has come when it is desirable to hold flood waters for periods of drought. It is hoped that the engineers of the U. S. Reclamation Service will be able to proceed with their policies of building dams along the Lower Colorado River for the impounding of flood waters for irrigation, land protection, and power development. Private capital is willing and ready to undertake these necessary enterprises in case Congress does not see fit to make the appropriations for their construction. These reservoirs are essential to the reclamation of the deserts and to the protection, both from flood and famine, of the 50 000 people who are already settled there.

If Congress does not soon act favorably on the recommendations of the engineers of the U. S. Reclamation Service pertaining to the Colorado River,

* Continued from December, 1922, *Proceedings*.

† Cons. Hydr. Engr., Los Angeles, Calif.

as outlined and referred to by Arthur P. Davis,* Past-President, Am. Soc. C. E., and under such Federal regulations as those described by O. C. Merrill,† M. Am. Soc. C. E., the Government should authorize others to proceed with these great reclamation works under a plan for construction and operation, supervised and approved by the officers of the Federal Government.

C. S. Jarvis,‡ M. Am. Soc. C. E.—After one of the meetings of the Colorado River Commission, during which one of the members held views divergent from the majority, some one spoke of the proponent of the minority opinion as "the stubbornest man in seven States". He had held out for unrestricted development of irrigation projects near the head-waters of the stream, whereas the prevailing view favored the priority of filings as a basis of rights.

The basis of the minority opinion is found in the following claims:

First.—Reduction of the flood crest, with resulting benefits to the lower projects now protected by levees.

Second.—Reduction of both the total sediment load and the percentage of transported material.

Third.—Increase of humidity in the drainage basin, with a consequent increase of precipitation and reduction of evaporation losses along the course of the stream.

Fourth.—Return flow representing a considerable proportion of the diverted waters.

In several instances, where investigations have been possible, the return flow in the Colorado Basin and adjacent territory has amounted to 30, or even 55 per cent. Where the lesser percentage applies, it seems apparent that an irrigator who diverts 1 000 acre-ft. from the flood crest, or surplus flow, effectively stores in underground reservoirs 300 acre-ft., part of which will contribute to the river during its low stages, and thus directly benefit the lower projects.

The lowest flow of the Colorado River has been completely appropriated, so that future development of the water resources in this Basin must be supplemented by storage. What is more effective storage than the underground reservoir under each irrigated tract in the uplands of this Basin? Underground reservoirs retain the water without the usual evaporation losses to which open reservoirs are subjected, and if their supply should be interrupted, they will continue to contribute for months, or even years, to the continuous flow. It is not unusual to note places in the Colorado River Basin, where a series of dry dams, maintained in a creek or river channel, successively divert the water supply for as many projects; and in extremely dry seasons, the lowest diversion is probably the most dependable supply, owing to the return flow from underground reservoirs.

The projects on the head-waters of the Colorado River, naturally, will develop more slowly than those in the lower valleys, owing to the difference in

* *Proceedings*, Am. Soc. C. E., November, 1922, p. 1738.

† *Loc. cit.*, p. 1758.

‡ Capt., Corps. of Engrs., U. S. A., Salt Lake City, Utah.

climatic and soil conditions, the cost of diversion from the canyons and gorges, and the inaccessibility of the mountain districts. Would any one, therefore, retard the natural rate of development of these projects by denying them a right to the primary flow of the stream when their diversion of flood waters both protects and supplies the lower projects?

Of course, the speaker's judgment is that the minority view is worthy of most serious consideration, and that the most logical development of production in minerals, in livestock, in agricultural products, in storage water for the lower projects, and the best growth of markets for power and commercial products, will follow an unrestricted irrigation development in the uplands of the Colorado River Basin.

D. C. HENNY,* M. AM. SOC. C. E.—The Columbia River offers large opportunities for the development of power, which, as far as the main stream is concerned, are as yet practically untouched.

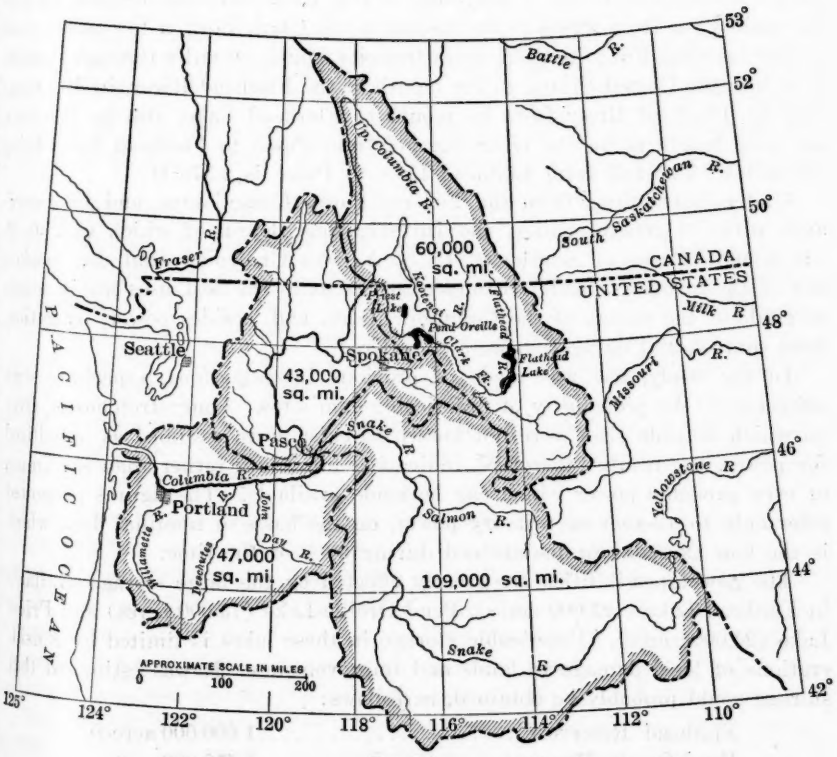


FIG. 25.

The drainage area at its mouth (Fig. 25) is 259 000 sq. miles, and its mean flow is 210 000 sec-ft. Table 16 permits comparison with a few other well known rivers.

* Cons. Engr., Portland, Ore.

TABLE 16.

Stream.	Point of measurement.	Drainage area in square miles.	Mean flow in second-feet.	Mean flow, in second-feet, per square mile of drainage area.
Colorado River	Mouth	242 000	24 000	0.10
Yukon River	Eagle, Alaska	254 000	73 000	0.35
Missouri River	St. Charles, Mo.	527 000	99 000	0.18
Nile River	Assouan, Egypt	1 262 000	115 000	0.11
Columbia River	Mouth	259 000	210 000	0.81
Danube River	Mouth	300 000	235 000	0.78
St. Lawrence River	Ogdensburg, N. Y.	383 000	249 000	0.65

The power possibilities, in conjunction with other uses, which may be made of the water of the Columbia River were studied recently by a Board of Engineers appointed by the Federal Power Commission. These studies were confined to that part of the river above the mouth of the Snake River which enters the Columbia near Pasco, Wash., and, of this part, there was considered only the main river from Pasco to the mouth of the Clark Fork, a few miles north of the Canadian line; the Clark Fork from its mouth, 20 miles through Canada, then into the United States, to the mouth of the Flathead River, in Montana; and the Flathead River from its mouth to Flathead Lake, also in Montana, the total length along the river channel from Pasco to Flathead Lake being 740 miles. The fall from Flathead Lake to Pasco is 2 570 ft.

Under instructions from the Federal Power Commission, and in accordance with its general policy, the Investigating Board, of which Col. J. B. Cavanaugh, Corps of Engineers, U. S. A., was Chairman, and the speaker, one of the members, made recommendations as to the best uses which might be made of the waters of the Columbia River, and, besides power, irrigation, flood control, and navigation were considered.

In the study, use was made of previous investigations, especially with reference to the probability of practicable dam sites. Long stretches of river on which feasible sites were not known to exist, were assumed as unutilized for power, the results, therefore, indicating minimum rather than maximum or even probable power which can be made available. The figures presented refer only to all-year or primary power, on the basis of nominal flow, which is the flow exceeded or maintained during 90% of the time.

The power possibilities are greatly affected by the large storage available in Flathead Lake (125 000 acres), Pend Oreille Lake (75 000 acres) and Priest Lake (24,000 acres). Practicable storage in these lakes is limited by considerations of local damage to lands and improvements. It was estimated that storage could probably be obtained, as follows:

Flathead Reservoir	1 600 000 acre-ft.
Pend Oreille Reservoir	1 750 000 "
Priest Lake Reservoir	600 000 "
Total	3 950 000 "

Including damages, the storage given, can be secured at low cost, probably between $1.00 and $1.50 per acre-ft.

The power possibilities were determined on the principle that all irrigation should have preference as to the use of the water, and allowance was made for the probable future diminutions of flow for this purpose.

As to irrigation, the one outstanding possibility is the reclamation of the Columbia Basin, in the State of Washington, which contains between 1 500 000 and 2 000 000 acres of irrigable land. The project has been investigated by the State of Washington, two diversions having been considered. One is a gravity diversion just across the State line in Idaho, the other is a pumped diversion at the head of Grand Coulee, the pumping to be done by power developed at an excellent dam site, which has been tested by deep preliminary borings.

These two diversions are mentioned, because a choice between them materially affects the power possibilities. The Board found that the power possibilities aggregated, as follows:

Streams as at present, no storage control..	1 725 000 primary e.h.p.
All future irrigation allowed for and full storage utilized:	
Columbia Basin irrigated by pumping...	2 443 000 " "
" " " " gravity.....	1 796 000 " "

The primary power given as available with a Columbia Basin pumping project is additional to the large amount of seasonal power required for pumping.

The large power possibilities of Columbia River as given, deal only, as previously stated, with the main river as far as Pasco. Below Pasco, it is possible that 150 000 and 500 000 primary e.h.p. may be made available, respectively, at Umatilla Rapids and Celilo Falls, which may increase the total main river power to about 3 000 000 h.p. A considerable part of the power in tributary streams like the Spokane, Yakima, and Wenatchee Rivers has already been developed, and large additional possibilities remain in these and other streams, notably the Deschutes, Chelan, and Kettle Rivers, all of which is independent of the great potentialities on the west side of the Cascade Mountains.

F. H. TIBBETTS,* M. AM. SOC. C. E.—Mr. Matthes has shown so clearly the scientific application of aerial photography to accurate mapping, that the many uses of simpler airscapes for illustrative purposes are in danger of being overlooked. Aerial photography can be of much assistance in many widely divergent lines of engineering work, both in the field and in construction. Photographs for this purpose are of two general types: Vertical views for aerial mapping, and oblique views ("bird's-eye views") for descriptive aids.

The vertical photographs are taken usually from an altitude of 10 000 to 12 500 ft., with the camera pointing vertically downward. Generally, they have been used in the construction of mosaic maps for military purposes. Maps of the same type have recently become quite common, especially for city mapping as an aid to the study of transportation, traffic, fire hazard, and similar problems, and for land development, to illustrate cultural conditions such as

* San Francisco, Calif.

areas in swamps, improved lands, brush, trees, etc., and to show the position and extent of existing improvements, roads, drainage, buildings, etc.

Work of this kind requires, however, special and precise photographic apparatus not commonly available in the United States, except through co-operation with the military authorities, and is usually too expensive for any except important projects.

Oblique or Bird's-Eye Views.—The oblique, or bird's-eye views, can be taken by any amateur with ordinary commercial apparatus. Photographs of this kind are of great value in illustrating preliminary reports, enabling other engineers who have not seen the territory to be covered, and particularly non-technical men interested in such work, to visualize rapidly and accurately the proposed projects. Aerial photographs may also be of great value for illustrating progress reports. Executives, boards of directors, or public officials, are thus enabled to appraise correctly the progress being made in a fraction of the time required to read a progress report, and with much greater accuracy.

In a study of city traffic problems, etc., any practicing engineer may obtain photographs by means of which he can, at his leisure, count automobiles or other units of vehicular traffic, freight cars, boats or buildings, or other improvements. For proposed land development work, aerial photographs show topography in relief, a result not satisfactorily obtained by vertical views, and thus enable those who have not seen the territory to understand correctly such topography, especially in broken or mountainous country. Simple photographs of this kind also show clearly the extent of forest or other vegetation, water-courses, swamps, or lakes, as well as improvements, such as roads, etc.

Good aerial photographs can be taken with any ordinary hand camera that is fitted with a shutter capable of operating at a speed of $\frac{1}{200}$ sec. Under average conditions of good light, probably $\frac{1}{300}$ sec. is better, with which timing, an aperture of F-5.6 to F-8 will ordinarily give properly exposed negatives. Care should always be taken to hold the camera free from direct contact with any part of the aeroplane, otherwise vibrations of the machine are more directly transmitted to the camera, with a corresponding loss in sharpness of the negatives. A camera with a focal plane shutter, will probably give better results and a sharper negative than one with the more common, between the lens, type of shutter. Ray filters will seldom improve results, even on a hazy day. The lightest ray filter in ordinary commercial use requires about three times the normal exposure, and under conditions of light that would indicate its use for ground work, an ordinary lens, without the filter, would be nearly full aperture with an exposure of $\frac{1}{50}$ sec. The use of a filter to time the negatives properly under these conditions, would require an exposure so long as to preclude clear negatives. Because they are less sensitive to the vibrations of the machine, plates probably give better results than films, although they are more difficult to use.

With aeroplanes of the more common types, the engineer or photographer occupies one cockpit, with his view somewhat obstructed by the wings or tail of the machine. To obtain good photographs an aviator of experience in this kind of work is required, or one who has been instructed in advance to

maneuver his machine so that the object to be photographed can be taken with the camera pointed diagonally away from the machine, about half way between the wings and the tail. The photographs should be taken when the machine is moving straight ahead, as clearer views will thus be obtained. The photographer, by pointing, can indicate the object that he wishes to photograph, and the aviator should then bring his machine into proper position. Ordinarily, the best results can be obtained from an altitude of about 1 000 ft. above the ground surface. The limits for such views, if much detail is desired, are probably between 500 and 1 500 ft.

The direct benefits to the practicing engineer of using an aeroplane for reconnaissance work should never be overlooked. Most engineers have had considerable experience with the use of the camera, and when work of this kind is to be done under their direction, can just as well take the photographs, being the best judges while in the air, of the subjects which they wish to photograph, and, at the same time, secure the benefit of direct inspection and study of the territory in which they are interested. When an idea of a rough mountainous area is desired, for example, more information can be obtained in a few hours in an aeroplane than in as many weeks of laborious and costly travel on the ground.

The speaker suggests as a matter of general policy that an engineer's duty is ordinarily only half completed when he has obtained a correct solution of the technical problem presented. The remainder of his duty to himself, and particularly to the public, is to convince non-technical, but vitally interested, persons of the merits of the work which he recommends. It is also his duty to keep the public interested and informed concerning the progress of construction work in which it is vitally interested. Aerial photography furnishes one of the most promising fields for informing, educating, and interesting the public in great enterprises.

Louis C. Hill,* M. Am. Soc. C. E.—Aerial surveys are of benefit when they are supplementary to or supplemented by comprehensive ground surveys. Under many conditions, however, the aerial survey can be of estimable worth when made independent of any ground survey beyond the necessary horizontal control.

In the study of the growth of cities and of the factors affecting congestion, the great detail shown on aerial photographs can be only little more than that indicated on maps of other types. The recent aerial survey of the business district of Los Angeles, Calif., is a fine example of the results that can be secured. The speaker is informed that remarkable accuracy was obtained.

The real value of the aerial survey is apparent in the study of areas covered by heavy brush, or that are otherwise inaccessible. Under the speaker's direction, such a map was made of the Kern River just north of Bakersfield, Calif., where topographic conditions are such that some form of protection from overflow is necessary to prevent periodic inundation of that city. The overflow areas were covered with dense brush. The channel of the river was divided by many low islands and, in many places, these channels were restricted so as

* Cons. Engr. (Quinton, Code & Hill), Los Angeles, Calif.

to be almost unrecognizable from the ground. Former channels of which no apparent trace remained, were clearly discernible on the aerial map. The position and magnitude of the canals, the banks of which form the present system of protection, were shown in relation to the various river channels. The effect of obstructions in the river, such as bridges and diversion weirs, became immediately apparent, as well as the amount and character of development on the areas to be protected. All this was shown as it actually existed, and none of it was affected by that indefinite factor, the personal equation. As a consequence, only a relatively small amount of ground surveying had to be done. Horizontal control was made sufficient to fix the scale of the aerial map, and levels were taken along lines projected on that map. Little more detail was needed to supplement the aerial survey.

Another survey is proposed of a large reservoir in an inaccessible region where the attitude of the Indians is such as to make surveying an extremely hazardous undertaking. Certain topographic maps were made several years ago of the probable dam site and of the lower elevations of the reservoir. It is felt that these maps can be extended quite accurately from an aerial survey. This is expected to be especially valuable in the study of the topographic and geologic conditions affecting the location of the dam and the spillways.

C. H. Birdseye,* Assoc. M. Am. Soc. C. E.—Mr. Matthes has very properly brought out the point that a rigid application of the method of utilizing aerial photography is not suitable to the survey of a large area, such as a "quadrangle" as mapped by the U. S. Geological Survey. The topographic maps of the Geological Survey are made in quadrangular units, each of which is an integral part of the map of the United States. These parts must fit together perfectly, and cumulative errors, such as those that are bound to result in surveys not based on accurate geodetic control, cannot be allowed. Since the World War, the U. S. Geological Survey has been using airplane photography as an aid to ground surveys to the fullest extent that its funds and the assistance placed at its disposal have permitted. Its principal difficulty has been in getting the Air Service of the Government to photograph the areas under survey. Limited personnel and equipment have prevented these services from giving the Geological Survey their fullest co-operation. The methods used by the Geological Survey have differed somewhat from those described by Mr. Matthes, but they have been no less successful and have resulted in a large saving in the expense of ground surveys.

Airplane photography was utilized in surveying the Schoolcraft Quadrangle, in Michigan, where 273 photographs, covering the area of 220 sq. miles, were taken by the Air Service of the Army. The total time consumed was 5 days, including the round trip from McCook Field, Dayton, Ohio. The actual photographic time, however, was only 7 hours, and the total cost of the photographic work, exclusive of the salaries of the officers, was $712 or $3.50 per sq. mile.

The ground control had already been established, and the base map projection was made, on which the control points were plotted and the land-line

* Chf. Topographic Engr., U. S. Geological Survey, Washington, D. C.

network adjusted before the photographs were used. Herein lies the difference between the method used by the Geological Survey and that of Mr. Matthes.

The photographs were joined together into small mosaics, about four pictures to a mosaic. All features on the photographs that would aid the topographer, such as roads, houses, streams, swamps, fence lines, and timbered areas, were inked on these small mosaics. The mosaics were then bleached, leaving only the inked lines on the scale of the original photographs, that is, 1 : 15 000. These bleached photographs were then reduced by photography to the scale of the base map, that is, 1 : 48 000. These reduced photographs were then adjusted to the control and the land-line net laid down on the base map, fence lines being registered over the section lines plotted on the map. Except at a few points, a perfect adjustment was obtained. The resulting combination of small photographs was re-photographed and printed in non-photographic blue on field plane-table sheets and sent to the field for contour sketching by the usual ground methods.

The cost of preparing these field sheets was about $1.50 per sq. mile, making the total cost of the photographic work less than $5 per sq. mile. The wealth of detail shown in the photographs, such as drainage, fence lines, and timber lines, enabled the topographer to make a great saving in the cost of the ground survey, which was $20 per sq. mile. The cost of office drafting was $3 per sq. mile, making the total cost of the survey $28 per sq. mile. Similar work in adjacent areas by the usual ground-survey methods cost about $40 per sq. mile, which indicates a saving of $12 per sq. mile, or 30 per cent.

The U. S. Geological Survey has also utilized airplane photographs in making base maps for the U. S. Bureau of Soils, in extensive surveys of the coast in Santo Domingo and Haiti, and in the revision of old topographic maps, using somewhat the intersection methods described by Mr. Matthes, except that the photographic control obtained from the photographs is tied to carefully executed ground control. The work in Santo Domingo and Haiti was done in co-operation with the Navy Department and the Military Government, and a reconnaissance map of the entire coast of Haiti has been prepared from airplane surveys, with no other ground work except the primary triangulation.

Airplane photography in topographic mapping will prove of greatest value in the revision of old surveys. It is obvious that a more accurate and complete revision of the culture can be made by the use of airplane photographs than by additional field work, which is likely to be only fragmentary.

Efforts are being made to develop a process by which relief can be shown accurately by means of the data obtained from airplane photographs. European scientists and the experts of the U. S. Army Air Service at the Experiment Station at McCook Field are working on this problem, but so far with very little practical result. Such application of photographic data may yet be made, but many years of research and experimental work will be necessary before a method can be developed that will be commercially feasible.

In recent years, many articles on airplane photography have been published, and the layman has been led to believe that all that is necessary tc obtain an authentic map of a region is to take a few airplane photographs and

paste them together. Such an operation will give a distorted photograph of the ground, which is properly called a mosaic. It is, however, not a map in any sense of the word, and unless the photographs are rectified and tied to well-established ground control, serious errors in both azimuth and distance will result. Nevertheless, airplane photography is the most valuable aid to topographic mapping that has been devised since the plane-table was perfected, and the U. S. Geological Survey proposes to use it to the fullest extent and to assist as much as possible in the development of new methods and new instruments.

EDWARD H. SARGENT,* ASSOC. M. AM. SOC. C. E. (by letter).†—Mr. Hogan is to be congratulated for his able résumé of the water power situation in New York State. It may be pertinent to note the availability of the extensive data obtained as a result of the investigations by various State departments during the past fifteen years.

In 1907, John R. Freeman, President, Am. Soc. C. E., began the first comprehensive study of the water powers of the State, at the direction of the State Water Supply Commission. However, the late George W. Rafter, M. Am. Soc. C. E., as early as 1895, made elaborate studies of the needs and effects of storage reservoirs, tentatively locating many reservoir sites. The investigations begun by the State Water Supply Commission in 1907 were continued until 1911, when that Commission was succeeded by the Conservation Commission. The Conservation Commission continued the work until 1921, when the work was transferred to the Department of the State Engineer, which has since continued the investigations, using therefor appropriations made to the New York Water Power Commission.

The results of the surveys, gaugings, borings, etc., made in the course of these investigations, have been embodied in many reports and pamphlets, most of which are available. Moreover, all the maps, designs, and estimates which have been made, are on file in the office of the State Engineer, and may be examined by any one interested. As a result of the investigations by these various State departments, it is believed that the development of many water power and storage projects will be facilitated.

It is stated‡ that the Indian Lake and Salmon River Reservoirs "are the only storage reservoirs of any size, that have been constructed in New York State for power purposes." Mr. Hogan neglected to include the Cranberry Lake Reservoir with a capacity of 3 000 000 000 cu. ft., which is on the East Branch of the Oswegatchie River. This was the first large storage reservoir built for power purposes in New York State, it having been constructed in 1867 by a commission of power owners. Perhaps, mention also should be made of the Delta and Hinckley Reservoirs which, although constructed primarily for water supply to the Barge Canal, nevertheless are operated so as to exercise a great beneficial effect on the water powers of the Mohawk River and West Canada Creek. These reservoirs have capacities of 2 750 000 000 and 3 400 000 000 cu. ft., respectively.

---

* Chf., Bureau of Water Power, Dept. of State Engr., Albany, N. Y.

† Received by Secretary, November 11th, 1922.

‡ *Proceedings*, Am. Soc. C. E., November, 1922, p. 1746.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

ENGINEERING GEOLOGY OF THE CATSKILL WATER SUPPLY

Discussion*

BY MESSRS. WILLIAM W. BRUSH, THOMAS H. WIGGIN, and JOEL D. JUSTIN.

WILLIAM W. BRUSH,† M. AM. SOC. C. E.—This paper is so complete that it does not leave much to be said in discussion, unless one goes outside the subject presented.

The speaker had the good fortune to be in charge of the studies for the City Aqueduct of the Catskill System. Extensive boring investigations were made so that these studies might be as complete as possible. In New York City, Dr. Berkey gave valuable assistance in determining the alignment of the pressure tunnel of the City Aqueduct. Owing to the location of this tunnel and the character of construction adopted, the traffic and life of the City were virtually unaffected during its construction. This is in striking contrast to the conditions in the streets when the many large pipe lines were laid for delivering the water to the smaller distributing mains.

Great difficulty was experienced in obtaining precise information regarding the rock formation underlying New York City from borings which had been previously taken, as well as from other excavations. The borings made by the Board of Water Supply have been so recorded and the rock samples so marked that no difficulty should be experienced by those who, in the future, may wish to use them.

Dr. Berkey developed a map of the geology of Greater New York from information that he had obtained before the borings of the Board of Water Supply were commenced. The location of the various rock formations shown on this map differed materially from those that had been given on other maps. The borings subsequently made under Dr. Berkey's supervision for the Board of Water Supply, proved the accuracy of his deductions, although,

* Discussion on the paper by Charles P. Berkey, Esq., and James F. Sanborn, M. Am. Soc. C. E., continued from December, 1922, *Proceedings*.

† Deputy Chf. Engr., Bureau of Water Supply, Dept. of Water Supply, Gas and Electricity, New York City.

in many instances, he had meager data from which to draw such conclusions. As a result of the work of the geologist, virtually no unnecessary borings were made.

The lesson to be drawn from the work on the City Aqueduct, as well as that on other parts of the Catskill System, is that the co-ordination of geological and engineering investigations can be used to great advantage and that such co-ordination should be utilized more frequently than has been the practice.

An experience in connection with the water supply system of Brooklyn is pertinent. The insistence on the part of certain persons that the water supply under Long Island comes from some drainage basin on the mainland and passes under Long Island Sound, caused the authorities of New York City to expend a large sum on developing a water supply based on that theory. Two pumping stations were built under a contract calling for the payment for water delivered, and both these stations have been abandoned. If the advice of a competent geologist had been sought, he would have supported the contention of the engineers, that on account of the geological formation an extensive water supply could not be transported from the mainland to Long Island.

The result of the combined study of the geologist and the engineer in the location and design of the City Aqueduct has been satisfactory as to operation. The speaker has been connected with the operation of the System since it was placed in service in 1917. The pressure tunnel which extends under the Bronx and Manhattan, to the two terminal shafts in Brooklyn, has met all expectations in delivering a large volume of water without serious leakage. Two or three small leaks have developed in the basement of buildings, but the flow from these leaks has decreased each year, and it has not been serious at any time.

THOMAS H. WIGGIN,* M. AM. SOC. C. E.—The Engineering Profession is indebted to the authors for the record of the geological investigations made for the Catskill works. They have properly emphasized the perfect co-operation which existed between the engineers and the geologists and which makes this work a good example of the proper method of conducting such investigations. Recognition is also due to the broad and economic policy of the former Chief Engineer, J. Waldo Smith, M. Am. Soc. C. E., supported by the Commissioners of the Board of Water Supply and by the engineering advisory staff, which made such a thorough investigation possible. Mining engineers have long made good use of geological investigations and core borings. Engineers in other branches have also made increasing use of these aids, but have generally confined this practice to special problems.

One of the earliest examples of a thorough geological investigation was that made for the Wachusett Dam, in Massachusetts, by Professor W. O. Crosby, for the late Frederic P. Stearns, Past-President, Am. Soc. C. E., then Chief Engineer of the Massachusetts Metropolitan Water-Works and afterward Consulting Engineer on the Catskill works. This investigation

* Cons. Engr., New York City.

was made a few years prior to 1900. On the same works, there was a tunnel the investigations for which were less complete. The portal was in rock and it was assumed that, farther on, the rock was higher and that, therefore, the tunnel would be in rock throughout. As a matter of fact, the rock was lower beyond the portal and for about 1 000 ft. remained at approximately mid-height of the tunnel, thus greatly increasing the cost and making an allowance to the contractor necessary in order to enable him to finish the work.

The so-called New Croton Aqueduct, beneath the Harlem River, penetrated rotten rock in a narrow crush zone and had to be placed at a lower elevation. As regards the number and the location of the borings, the rock had been well explored. The importance of the narrow crush zone, however, as it was apparently disclosed by the boring which traversed it, was underestimated as could easily be done under the circumstances. The disaster that occurred in 1908 when the Loetschberg Tunnel, in Switzerland, was under construction, is still comparatively fresh in the minds of engineers. Owing to reliance on prognostications of geologists, unconfirmed by borings, a zone of disintegrated, wet, and flowing rock débris, thought by the geologists to terminate well above the tunnel, was unexpectedly penetrated and the tunnel was filled so quickly that 25 men were killed.

The Catskill works had all these examples and others from which to profit and they were heeded. It was realized that a new element was injected into the investigations by the unprecedently high unbalanced pressure of 200 to 400 ft. that would exist in the pressure tunnels when they were in service. The speaker had charge of the design of these tunnels and it was his privilege, in the early stages of the geological investigations, to aid the geologists in visualizing these unusual conditions which differed so greatly from those in the ordinary mining or academic problems.

The Rondout Pressure Tunnel presented one of the most complicated geological problems. Preliminary estimates indicated that the cost of the pressure tunnel would be about the same as that of steel pipes. As it developed later, however, steel pipes would have been somewhat cheaper and, if the Catskill Aqueduct type of concrete-covered, mortar-lined, steel pipe, later suggested by the speaker and used for siphons where the geology was unfavorable for tunnels, had then been evolved, it is possible that such pipes would have been chosen instead of the tunnel, as the tunnel was expected to be wet in the section penetrating the Binnewater sandstone and High Falls shale. There is no reason to regret, however, that the more conveniently operated and probably more permanent tunnel was built. The geologists and the engineers were brought into definite contact first in studies of the Rondout Pressure Tunnel when the attempt was made to translate the generally qualitative conceptions of the geologist into the quantitative results necessary to estimate the quantity of water to be pumped during construction, lengths of tunnel that would require timbering, possibilities of outward leakage, etc. The stratification of the rock shown on the tunnel profile, which formed the basis of the studies for depth of tunnel, was mainly the result of close co-operation between the Field Division Engineer, Lazarus White, M. Am. Soc. C. E., his Principal Assistant, J. P.

Hogan, M. Am. Soc. C. E., and Professor Crosby, Consulting Geologist, in placing and interpreting borings. Professor Crosby and the speaker subsequently discussed the factors affecting the proper depth of tunnel. Professor Crosby's long experience in mines gave him a definite conception as to the probable leakage, and as the result of this conference, a part of the tunnel was placed 100 ft. deeper than had been previously contemplated. The rate of leakage into this tunnel on the incline where the porous Binnewater sandstone and High Falls shale were penetrated, was somewhat underestimated, but the difficulty in removing the water and of obtaining a tight tunnel was perhaps over-estimated. After the experience with wet ground in the Rondout Tunnel, the pressure tunnels to be built subsequently were viewed with less apprehension both by the engineers and the bidding contractors.

Certain pure limestone strata traversed by the Rondout Siphon caused anxiety to the designers, on account of the experience in the Thirlmere Aqueduct, in England, where the concrete made of similar limestone dissolved badly. As the authors state, this limestone was not used in the concrete lining. The rock through which the tunnel passes is still subject to solution, but reliance was placed in a good lining and careful grouting to reduce the circulation to a negligible quantity. Furthermore, the small carbon dioxide content in the water and its reduction by aeration at Ashokan Reservoir will minimize the power of the water to dissolve the limestone. The tunnel is becoming tighter all the time.

The Wallkill Siphon offered no geological difficulties. It was entirely in shale and rightly expected to be largely free from inward leakage. After the rock profile had been determined by a sufficient number of borings, the only important problem was the necessary depth of tunnel below the rock surface. This was a pioneer line of thought with respect to such large unbalanced pressures.

One criterion applied by the designing engineers was that the depth should be such that the weight of earth and rock above the tunnel would offset the upward unbalanced water pressure, estimated as the height of a column from the hydraulic gradient to the level of ground-water. This condition, however, might be met by a deep cover of earth and a shallow cover of rock. Stability and water-tightness demanded a substantial thickness of rock; even a thick rock cover had its possible weakness, as a laminated rock could conceivably fail by the accumulation of full pressure in a horizontal seam near the top. A failure of this kind occurred later at the shaft of the City Tunnel in Bryant Park, New York City. The minimum depth of pressure tunnel below the top of sound rock was decided as 150 ft. This was an arbitrary thickness suggested by the speaker and accepted by the Chief Engineer and his advisory staff, and, thereafter, became a standard for fixing the depth of other pressure tunnels on the Catskill System, including the City Tunnel.

Moodna Siphon was partly in shale generally like that penetrated by the Wallkill Siphon and partly in granodiorite like that through which the

Hudson Siphon was constructed. As described by the authors, the stretch in shale was suspected of harboring a buried gorge of an ancient stream. Many borings were made in the search for that gorge, some of which were only about 150 ft. apart. Luckily no gorge existed, although the reason that the ancient stream left such a broad flat floor in such easy shale digging is still somewhat puzzling.

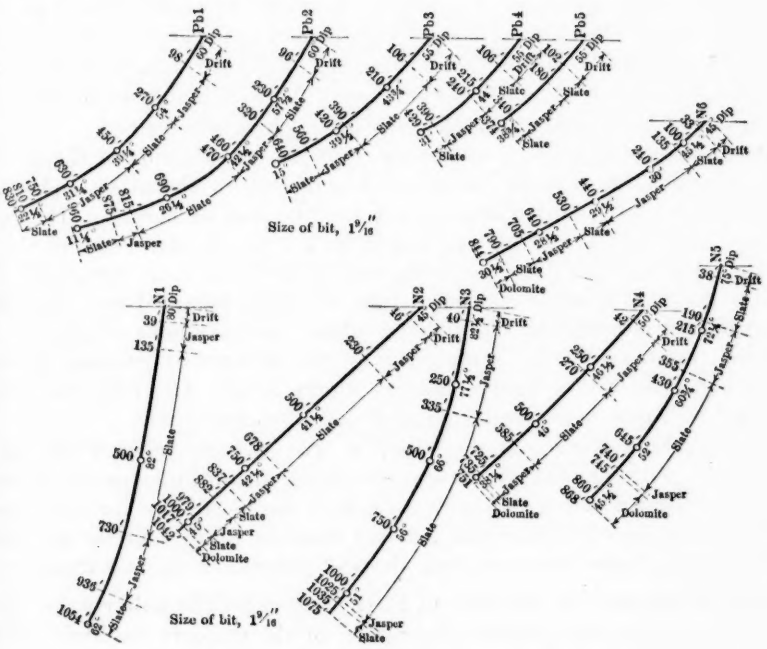


Fig 19.

Borings in the river to discover the Hudson Gorge were unsuccessful, although one hole, beginning with an 18-in. and ending with a 2-in. casing, penetrated more than 760 ft. It took more than a year to put this boring down, and it petered out as boulder after boulder required successive reductions in the size of the casing. At about that time, the speaker was sent West to investigate inclined shafts in the Michigan copper country. This investigation was apropos of the proposal to sink an incline from each side of the Hudson River toward the center, maintaining each incline at a safe distance below the rock surface by making pilot borings from the incline ahead to the rock surface. The speaker gained considerable information about long inclined borings, and it was decided, instead, to make such borings from the vertical shafts already started. Fig 12* shows how this was done, although it does not explain why two sets of borings were needed. The records of long inclined borings for Michigan mines showed that they curved upward considerably, as shown by Fig. 19. It had been found that close clearances

* *Proceedings*, Am. Soc. C. E., September, 1922, p. 1573.

in the first two boring rods, that is, a small difference between the size of hole as determined by the setting of the diamonds and the diameter of rods, high rotary speeds, and light rod pressure resulted in less curvature, but did not eliminate it. In the specifications for the long inclined Hudson borings, a generous size of hole and all the previously mentioned aids to straightness were used. The lower holes shown in Fig. 12 were drilled first. They were expected to curve upward so as to occupy, at the center of the river, a position not much deeper than that explored by the second or upper set of holes. They remained practically straight, however, and went too low, and the upper pair of holes, therefore, was bored to explore the rock higher up and to allow the tunnel to be located at a higher elevation, which was done.

Another consideration in connection with the investigation of the Hudson Siphon, which, in a lesser measure, is pertinent to the other siphons, is the possibility of a buried falls just below the line of borings. It is conceivable that a thin wall of rock might be left on the down-stream side of the tunnel at such a falls, even if the vertical rock cover was sufficient. The speaker does not recall that such a falls was then regarded as a geological impossibility, although he notes that evidence now available relating to the Hudson Gorge southerly, is regarded by the authors as proving an overdeepening effect rather than a falls at Storm King. The falls was always regarded as a remote possibility and did not affect the profile.

In describing repairs to the Eastview Tunnel,* the authors omitted to state that a water-proofing diaphragm, consisting of three thicknesses of cotton duck impregnated and coated with asphalt in the usual way for such water-proofing, was placed between the original concrete lining and the new brick lining. It was hoped thus to exclude the acid water from the new lining.

JOEL D. JUSTIN,† M. AM. SOC. C. E. (by letter).‡—The writer was particularly interested in the authors' description of the geologic conditions at the sites of the Gilboa and Devasego Falls Dams. It would be most interesting to learn more of the considerations which led to the abandonment of the Devasego Falls site, where topographical conditions indicated much cheaper structures than those required at the adopted Gilboa site.

The authors state that the ridge of glacial drift which forms in part a natural dam across the Valley of the Schoharie at this place, contains a pervious layer about 1 000 ft. wide, 1 200 ft. long, and 30 ft. deep. The top of this pervious stratum is stated to be at about Elevation 1 100.

Through the work of Messrs. Allen Hazen, M. Am. Soc. C. E., Slichter, and other experimenters, the laws of flowing water through pervious soils, sands, and gravels are quite well known. With a thorough sub-surface investigation, such as that at the Devasego Falls site, it is possible to predict, within reasonable limits, the leakage through such a stratum. It would also be possible to determine whether the velocity of any water that might issue to the surface at the down-stream end of the pervious stratum would be sufficient to move any of

* *Proceedings*, Am. Soc. C. E., September, 1922, p. 1583.

† Chf. Engr., Depts. of Hydro-Elec. Power Plants, Water Supply, and Sewerage, The Ludlow Engrs., Inc., Winston-Salem, N. C.

‡ Received by the Secretary, December 5th, 1922.

the constituent particles of the stratum. The value of the water lost by leakage through such stratum could also be determined. With the results of such a study available, it would be possible to determine whether this ridge of glacial drift could safely and economically be utilized as a part of the proposed earth dam, whether a cut-off through the pervious stratum would be desirable, and whether safety and economy required the abandonment of the site in favor of some other.

Apparently, the total head of water on the pervious stratum would not have exceeded 60 ft. Assume that the stratum extends from the up-stream face of the ridge through to the down-stream side, a distance of 1 200 ft. Also, assume that the effective size of the material is 0.10 mm., that the porosity is 32%, and that the ground-water temperature is 60° Fahr.

Slichter's formula* for the flow of water through sands and soils is, as follows:

$$q = K\frac{p\,s}{h}$$

in which,

q = discharge, in cubic feet per minute;
p = difference in elevation between the two points considered or difference in pressure measured in feet;
h = distance through soil along path followed by water;
s = area of saturated cross-section of soil considered; and
K = the transmission constant.

The factor, K, is always the same for the same soil at the same temperature. The transmission constant, as defined by Slichter, is "the quantity of water that is transmitted in unit time through a cylinder of the soil of unit length and unit cross-section under unit difference in head".

The proper transmission constant for the given effective size and porosity, originally determined experimentally, is 0.00405, as obtained from Table 1, of *Water Supply Paper No. 67*.

By making the proper substitutions in Slichter's formula,

$$q = \frac{0.00405 \times 60 \times 1\,000 \times 30}{1\,200}$$

$$= 6 \text{ cu. ft. per min.}$$

If instead of the effective size being assumed as 0.10 mm., it had been assumed as 0.35 mm. (about the size of filter sand), computations would have indicated a leakage of a little more than 1 cu. ft. per sec. under the assumed conditions.

The velocity of the water flowing through the soil would be obtained by dividing q by the area of the water passages, or,

$$V = \frac{q}{s\,P}$$

* *Water Supply Paper No. 67*, U. S. Geological Survey.

in which,

V = velocity, in feet per minute;
q = discharge, in cubic feet per minute;
s = area of cross-section of soil through which water is flowing; and
P = percentage of porosity expressed as a decimal.

Thus, with the computed discharge of 6 cu. ft. per min., $V = 0.00063$ ft. per min., or less than 1.0 ft. per day, which is less than that required to move any particles of the assumed material. These computations are merely illustrative. No doubt a study along the lines herein discussed was made by the engineers of the Board of Water Supply. It is to be hoped that the details of the investigation and study of the Devasego Falls site will be described, as they would be of great value to engineers having similar problems.

In some sections of the United States, impervious foundations for earth dams are not available and some earth dams must be built of quite pervious material. Where such conditions have been found and properly interpreted by thorough sub-surface investigations, safe dams have been designed and built.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

LOCOMOTIVE LOADINGS FOR RAILWAY BRIDGES

Discussion*

BY MESSRS. CHARLES EVAN FOWLER, C. F. LOWETH, RALPH MODJESKI, and C. A. P. TURNER.

CHARLES EVAN FOWLER,† M. AM. SOC. C. E.—A Composite loading was first proposed before the Society in the speaker's paper entitled, "Revision of the Niagara Railway Arch Bridge."‡ The same general idea is being used by P. B. Motley, M. Am. Soc. C. E., on the Canadian Pacific Railway, by J. J. Yates, M. Am. Soc. C. E., on the Central Railroad of New Jersey, and by others.

When the speaker broached this subject to the Committee of the American Railway Engineering Association in 1919, he found the members of that Committee so strongly in favor of retaining the Cooper Class E-loading that the matter was not pressed, as that Committee represented the engineers who are responsible for railway loadings. Thus, it is certain that the Association and the Engineering Institute of Canada must be consulted and first be satisfied if any proposed loading is to be substituted for the Cooper system, even though that is so badly out of date and out of time with modern bridge engineering.

The history of bridge loadings shows that changes must come about slowly. The speaker recalls the wheel load diagrams with resultant panel loads, published about 1885, by the late Thomas H. Johnson, M. Am. Soc. C. E., of the Pennsylvania Lines Southwest; the ones most used were two 74.5-ton engines followed by 32-ton cars, or about 1 640 lb. per lin. ft.; two 92.3-ton engines with the same following load; and, in 1888, the undreamed-of load of two 107.5-ton engines followed by 4 000 lb. per lin. ft. About this time,

* Discussion on the paper by D. B. Steinman, M. Am. Soc. C. E., continued from December, 1922, *Proceedings*.

† Cons. Engr., New York City.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1919.

J. E. Greiner, M. Am. Soc. C. E., devised the plan of using a slide-rule of special construction for sliding the train along panel pointers to obtain moments and shears. Soon after, it was discovered that equivalent uniform load curves or tables could be calculated and practical accuracy obtained as great as that by the laborious use of wheel loads. The engineers of the Youngstown Bridge Company constructed such curves for the locomotives of about thirty of the railways for which bridges were being built and no errors resulted. This method has largely continued to the present time, and it is little trouble to shift to uniform load curves of some type of Composite loading. What shall this be? Manifestly, if standard axle spacings and loads for the different types of locomotives could be adopted, as was considered by the speaker in his discussion* of the Tentative Specifications for Steel Railway Bridges, the problem would be simplified and the standard adopted would be more lasting. This would cover Class A for Mallets; Class B for the Santa Fé type; Class C for Mikados; Class D for electric locomotives; Class E for the Cooper loadings, and Class F for the final or Composite loading. (See Figs. 30 and 31 in the discussion mentioned.) Some of the road officials already have taken action in this direction and toward reducing counterbalancing, and the rapid change to electric operation being made on many roads will cause still further consideration. The slow speed of freight engines makes the effect of load plus impact no greater, perhaps, than that of high-speed passenger locomotives of lesser total weight. The heaviest locomotives are only used on a limited mileage, whereas on branch lines much lighter locomotives are used than on the main lines. The new Michigan Central locomotive, No. 8 000, will haul a train of 120 cars, but the car weights or train load per linear foot are no heavier, and doubtless seldom reach more than 4 000 lb. per lin. ft. This is one example of the fact that the train loads should be from 500 to 1 000 lb. per lin. ft. lighter for each Cooper loading after E-50 is passed. This and various other matters of loading are discussed in the speaker's paper on the Niagara Railway Arch Bridge.† Thus, the problem of devising a final Composite loading is somewhat complex, and must only be done by all those interested.

C. F. Loweth,‡ M. Am. Soc. C. E.—The speaker is in sympathy with the quite general desire to simplify and, at the same time, to make more practical the typical engine and train loadings used for proportioning railroad bridges.

When he first began his experience in designing railroad bridges, it was the custom to ascertain the heaviest locomotive and train loading in use anywhere and utilize them in proportioning the design of the proposed structure. Now and then, something was added for possible increase in weight of locomotives and train loadings. This method involved a great deal of painstaking work and resulted not infrequently in the boast that the structure was designed for the heaviest locomotive known; but often before the bridge was completed, a heavier locomotive was built, with altogether different axle

* *Proceedings*, Am. Soc. C. E., October, 1922, p. 1717.

† *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), pp. 1925 to 1930, and p. 2020 *et seq.*

‡ Chf. Engr., C., M. & St. P. Ry., Chicago, Ill.

spacings, and all the painstaking efforts for perfection of design to meet actual conditions were defeated. The author's suggestion that railroad bridges should be proportioned for a typical uniform loading would be ideal, if one could be determined that would approximately fit actual conditions and would be generally accepted and used. Such a practice would be along the lines of that of the noted bridge designer, the late George S. Morison, Past-President, Am. Soc. C. E. His method was to take a uniform train load, headed by a largely increased loading for a length of one or, perhaps, two panels, in order to allow for the additional weight of the locomotive over that of the train. The application of this loading was simple and had many advantages, but it fell into disuse, because it did not approximate closely enough actual conditions, and, perhaps, equally on account of the fact, that the loadings then generally used did not provide sufficiently for the increasing weight of locomotives and cars.

The speaker remembers the frequent arguments in those early years as to whether locomotives would continue to be built materially heavier than the largest in use at the time. Each new locomotive seemed to be about the maximum that could be designed and would be practicable; yet, as the years passed, there was a continuing succession of heavier locomotives, and the end is not yet in sight. With heavier rails and better track construction throughout, stronger equipment, mechanical stokers and other locomotive improvements, the increasing cost of transportation, and the necessity for the utmost economy in railroad operation, it is quite possible that locomotive and train loadings may continue to increase in weight.

However, railroad transportation is undergoing an evolution and the trend in the future may not be in the same directions as that in the past. The speaker thinks that it may be within the bounds of possibility that future transportation requirements may change, in some localities at least, to a demand for smaller train units, more quickly moved. This suggestion may smack of heresy, because to the present time, it seems to have been a generally accepted rule that economical railroad transportation requires train units as large as can possibly be handled.

Another possibility to be taken into consideration is that of the electrification of steam railroads. As the financial problem involved in electrification is so great, the speaker does not expect to see the change from steam to electric power, for general railroad purposes, come about rapidly. There will be, however, a tendency in that direction. The electric locomotives are generally lighter in weight, at least when measured by units of length, and, what is perhaps even more important, they produce less impact effect on track and structures. They undoubtedly will be less frequently double-headed for heavy trains than steam locomotives. These considerations tend to reduce the justifiable weight of train loadings.

It would be desirable if locomotive loadings could be safely standardized without getting too far afield from the actual conditions to which the structure would be subjected. A steel bridge superstructure, for the most part, is permanently located. There are parts of railroads where the traffic requirements will never call for locomotives and train loadings as heavy as those now in

general use. There are other parts where the traffic requirements will call for increasingly heavier train loadings. The speaker thinks that advantage of this difference in conditions is not as generally taken as it should be. He also questions whether sufficient consideration has been given the fact that the heaviest locomotives are those used in freight service which, from their design, as well as from their use, are limited to speeds much less than those of the lighter passenger locomotives. There are also other conditions which effect the speed of trains over any structure. Speed is a factor in the impact effect of the load, and to apply the impact for maximum speeds, irrespective of the structure and type of locomotive used, is not a rational method of design.

Relative to the effect of the electric locomotive on bridges on the Pacific Coast Extension of the Chicago, Milwaukee, and St. Paul Railroad, the speaker would explain that those bridges were designed before the decision as to electrification had been fully made. The bridges were proportioned for what is known as the Cooper E-55 loading. The electric locomotives in use, although among the heaviest, if not the heaviest, which have been built for any service, are well within the limits of the E-55 loading for which the bridges were designed. The electric locomotives have practically no unbalanced and reciprocating parts so that their impact effect on track and structures, as compared with steam locomotives, is small.

RALPH MODJESKI,* M. AM. SOC. C. E. (by letter).†—There is no doubt, and it has been well brought out by the author, that the Cooper standard loading does not properly represent modern loading conditions. It would be impossible to construct a rating diagram which would correspond to the power as well as to the equipment of every railroad in the United States. It has long been recognized that the Cooper diagram cannot be strictly applied to all loadings, principally because, in individual cases, the maximum wheel concentrations have no fixed relation to the train loads. Therefore, instead of calculating a bridge for the Cooper standard loading, some combined loading would be taken, as, for instance, the Cooper E-60 locomotive followed by a train load of 5 000 lb. per ft. of track. A combined loading of this kind may be fitted quite closely to the motive power and to the train loads of any railway system. It has the advantage of allowing comparison between bridges constructed at different periods; their relative rating may then be determined without extra calculations and may be easily understood by the railroad executives.

If a new loading is adopted at this time, the comparison of the rating of existing bridges that were calculated on the basis of the Cooper loading, with the new bridges calculated by such loading, would be obliterated and could only be made by the use of tables or calculations, or both.

Referring to Table 2,‡ it may be noted that the ratings for the center moments and end shears are quite consistent for spans up to about the length of two Cooper locomotives. Beyond that length the rating decreases, owing to the increasing influence of the uniform train load. For a 1 000-ft. span,

* Cons. Engr., Chicago, Ill.

† Received by the Secretary, November 9th, 1922.

‡ *Proceedings*, Am. Soc. C. E., May, 1922, p. 1050.

the ratings are practically equal to the uniform train load of the Cooper E-60 loading as well as to that of the proposed M-60 loading, because the influence of the locomotive load becomes small. If, therefore, all spans from 10 ft. to 1 000 ft., were calculated for a load consisting of the Cooper E-73 locomotives followed by 6 000 lb. per ft. (calling such a load E-73-6 000 for convenience), the ratings of Table 2 would become uniform for all practical purposes and would closely correspond to the author's M-60 loading. Some slight discrepancies or even inconsistencies might occur, but this would also be the case with any other system of wheel loading. Considering that unit stresses are based only on good judgment and, further, that allowances must be made for impact, unbalanced wheels, wind, tractive forces, and secondary stresses, the difference between some such loading as E-73-6 000 and M-60, will be small indeed.

If a revision of loading is advisable, would it not be better to adopt at once different classes of loading based on a uniform load and one or more "floating" concentrations, such as those proposed by the author as a second alternative, rather than resort to another system of wheel loading which may again have to be revised in the near future. The writer believes that a uniform loading with one floating concentration would fill all requirements. Some large bridges in the United States have been calculated in such a manner.

C. A. P. TURNER,* M. AM. SOC. C. E. (by letter).†—As a railroad train passes over a bridge structure, a certain element of time is required to overcome the inertia of the mass in producing the greatest deflections of the parts and of the structure as a whole. The greatest elongation under tension, or deformation under compression, is a thermo-dynamic function such that the mechanical energy represented by the transfer of the kinetic heat energy of molecular vibration in the resisting metal to the potential energy of deformation of strain is double the external mechanical work of the load when the stress is tension, and the reverse when it is compression. This excess energy is radiated to the surrounding medium or the absorption of heat energy from it takes place. Accordingly, there is an instantaneous or kinetic deformation and a static deformation which involves the thermal effect.

The relative amount of energy involved, as shown by Kelvin's thermal dynamic theory, is approximately double that of the work of the external applied forces. The full development of the static deformation requires an element of time, which in connection with overcoming the inertia of the mass, may be approximately estimated at 1 sec.

If the train is traveling nearly 60 miles per hour, its speed may be taken at 100 ft. per sec., and the vertical impact on a member would approximate,

$$S\left(\frac{100}{A+100}\right)$$

in which,

S = live load stress; and

A = length of track loaded, for maximum effect, when the initial dead load stress equals zero.

* Cons. Engr., Minneapolis, Minn.

† Received by the Secretary, November 16th, 1922.

However, if the member is under dead load stress, the approximate vertical impact strain would equal,

$$S \times \left(\frac{L}{D+L} \times \frac{100}{A+100}\right).$$

The rolling load tends to set up structural vibration laterally as well as vertically, and the total impact is the resultant of the vertical and horizontal effects combined. In the ordinary pin-connected bridge, with a train speed of about 60 miles or more per hour, the lateral impact may be assumed to be approximately equal to the vertical impact effect. Therefore, for the pin-connected railroad bridge, the formula for total impact is:

$$I = S\left(\frac{L}{D+L} \times \frac{140}{A+100}\right).$$

The stiffness of the riveted structure would reduce the necessary impact allowance to about

$$I' = S\left(\frac{L}{D+L} \times \frac{130}{A+100}\right).$$

The vertical effect of impact would vary approximately as the speed of the train, whereas the lateral effect would vary roughly as the square of that speed. Accordingly, if the rational impact allowance of the high speed passenger train is to be compared to that of the ordinary freight train, which travels about half as fast and the lateral impact of which would be one-fourth as great, these formulas would immediately reduce, for the pin-connected bridge, to

$$I = S\left(\frac{L}{D+L} \times \frac{55}{A+50}\right)$$

and corresponding values for the riveted bridge.

It accordingly appears that the value of the passenger train impact for a Cooper E-60 loading would be nearly equal to an E-75 loading for freight train speeds. As the excessive Mallet engines are not used for fast passenger traffic, there is little call for a radical change in the present typical engine loading when coupled with a rational impact formula. The present standard furnishes a uniform method of comparing the capacity of bridges, and tables for its convenient application are in general use.

The fact that the Cooper loading approaches as closely to the wide range of engine loading as any that can be devised and, if used in conjunction with a rational impact formula, meets all requirements, argues against introducing other and different types of typical engine loadings, which would only add confusion to present methods of investigation. If the impact formulas are considered for range of stress from tension to compression, the first member in the foregoing suggested formula would become unity and may be disregarded.

Mr. Steinman's painstaking investigation is a distinct addition to the literature of the subject, but his showing when considered in connection with rational allowances for impact does not indicate the need of any radical change in the well established Cooper loading.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

TESTS OF CONCRETE IN SEA WATER

TECHNICAL PAPERS PRESENTED AT THE ANNUAL CONVENTION, PORTSMOUTH, N. H., JUNE 21ST, 1922

Discussion*

BY D. E. HUGHES, M. AM. SOC. C. E.

D. E. HUGHES,† M. AM. SOC. C. E. (by letter).‡—Eight large series of tests of 1:3 sand briquettes were made years ago, on account of the San Pedro breakwater. All these tests were similar, except in kind of water used in mixing and in time of immersion. Series *A*, *B*, *C*, and *D* were mixed with sea water. After 24 hours, Series *A* and *B* were immersed, *A* in sea water and *B* in fresh water. Immediately on being made, Series *C* and *D* were immersed, one in sea water and the other in fresh water. Series *E*, *F*, *G*, and *H* were mixed with fresh water, *E* and *F* being immersed after 24 hours in fresh water and sea water, respectively, and *G* and *H* being placed immediately, *G* in fresh water and *H* in sea water. In all cases, the briquettes remained in their moulds 24 hours without being removed from the glass plates on which they were made.

All these briquettes developed practically equal strengths, except those of Series *H*, which, at every age and without exception, were decidedly weaker. The opinion was reached that, in Series *H*, mechanical interference with setting resulted from the sea water displacing the lighter fresh water, and this led to the advice to use sea water in the mixing of concrete that is to be exposed to sea water before setting. In large masses of mortar or concrete, the displacing of fresh water by salt water would be less complete and the weakening less pronounced.

Respecting consistency of concrete, the depth one sinks into fresh concretes, even of equal wetness, depends on whether there is much, little, or no excess

* Discussion of the paper by L. C. Wason, M. Am. Soc. C. E., continued from November, 1922, *Proceedings*.

† San Pedro, Calif.

‡ Received by the Secretary, November 20th, 1922.

of mortar, and whether the aggregate is of broken stone or the more easily displaced rounded gravel. Therefore, such expressions as "ankle-deep", etc., are hardly indicative of relative wetness, except in the case of concretes that otherwise are equal.

As to rusting of reinforcement, it can be stated that iron and steel long embedded in clay or mud under the sea have been found to be well preserved, because the first installment of water to reach them had only a limited effect, and the clay prevented displacements by succeeding installments to repeat the attack. Probably, the comparative freedom of reinforced concrete from cracking below low tide is similarly explained. At higher elevations, both gravity and evaporation operate to make room for new water to continue the rusting and swelling of steel.

In Southern California, the earliest failures are above high tide where concrete is reached only by salt air and spray and where no animal or vegetable matter or other scum attaches to seal the pores against renewal of waters.

The need of guarding against rock-borers may also be mentioned. In the office of J. W. Ludlow, M. Am. Soc. C. E., Chief Engineer of Los Angeles Harbor, there are specimens of concrete attacked by such borers. These specimens were obtained from concrete pile jackets placed 14 years ago and recently removed on account of channel widening at East San Pedro. These borers make small entry holes, but they immediately enlarge to 1 in. in diameter, and some of them have extended through the 3-in. jackets.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

ROBERT MOORE, Past-President, Am. Soc. C. E.*

DIED JULY 24TH, 1922.

Robert Moore, the son of Henry Clay and Amelia N. Whippo Moore, was born at New Castle, Pa., on June 19th, 1838. His great-grandfather, Dr. Henry Moore, came to the United States from Dublin, Ireland, in May, 1773, and settled in Washington, Pa. His maternal grandfather was Charles T. Whippo, an engineer employed on the construction of the Erie Canal, of Pennsylvania, who, later, became Chief Engineer of the Canal from Beaver to Erie, Pa.

Mr. Moore was graduated from Miami University, of Ohio, with the degree of B. A., on June 20th, 1858, later, receiving the degree of Master of Arts. His early engineering training was obtained directly under his father, who was a prominent civil engineer, engaged largely in the pioneer railroad development in the Middle West.

Mr. Moore's early manhood covered the period of the Civil War. In 1863, he was engaged as a Civilian Engineer with the Engineer Corps, U. S. Army, on fortifications in Kentucky. From 1864 to 1877, he was engaged in the location and construction of various railways in Ohio, Indiana, Illinois, Missouri, and Kansas, lines which now form part of the trunk-line systems of the United States.

With the adoption of a new charter by the City of St. Louis, Mo., in 1876, the Board of Public Improvements, which has charge of the engineering work of the city, was established in 1877. Mr. Moore was a member of the first Board, and acted as Sewer Commissioner from 1877 to 1881. His confrères on this Board were all eminent men in the Engineering Profession, namely, the late Henry Flad, Past-President, Am. Soc. C. E., Thomas J. Whitman, M. Am. Soc. C. E., Gen. John W. Turner, and Mr. Charles Pfeiffer. The capable and efficient handling of public affairs by this Board gave the Profession a standing and prestige in St. Louis, that has remained a factor in the government of the city to the present day.

Early in his professional career, Mr. Moore perceived the disadvantage under which a man labors, who becomes a one-company engineer, and, thenceforth, he devoted himself to consulting practice. The spirit of independence which this illustrates, was one of his most marked characteristics. Accordingly, from 1881 until his retirement from active practice in 1920, his engagements were very largely as a Consulting Engineer. During that time, he was connected with some of the most important construction projects and difficult problems of those years: He was in charge of the re-construction of the rail-

* Memoir prepared by the following Committee appointed by the Board of Direction: Messrs. Baxter L. Brown, *Chairman*, F. G. Jonah, and W. E. Rolfe.

way floor system of the Eads Bridge; Consulting Engineer of the St. Paul and Duluth Railway; Chief Engineer of the St. Louis Merchants Bridge Terminal Railway; and Consulting Engineer for the Re-organization Committees of the St. Louis Southwestern Railway, the Rio Grande Western Railway, and the Santa Fé System. He was Chief Engineer of the St. Louis, Peoria, and Northern Railway, a line extending from St. Louis to Peoria, Ill., from 1894 to 1898; and during this time, he also represented the Los Angeles Terminal Railway in the San Pedro-Santa Monica Harbor controversy. He was also Chief Engineer of the Wabash, Chester, and Western Railway, and the Missouri Extension of the Missouri and Illinois Railway. Mr. Moore was a member of the Brazos River Board in 1897, and a member of the Southwest Pass Board in 1899 and 1900. He also acted as a Special Consulting Engineer on the Missouri and Mississippi River Bridges, for the Chicago, Burlington, and Quincy Railway, from 1900 to 1920.

In addition to his professional work during these years, Mr. Moore found time to take a most active part in the educational affairs of the City of St. Louis. He became a member of the Board of Education in 1897, at a time when the public had been aroused with respect to the evils existent in the public school system, and served as a member of what was then designated as the Reform Board, composed of twelve members, which took the place of the old system of ward representation. At that time, Mr. Moore had already reached an age when most men, although conscious of civic duty, feel entitled to be relieved from such demands on their time; however, he not only acted through his original term of election, but through successive terms, continuously, for sixteen years. He was President of the Board of Education in 1906 and, again, in 1910, and during practically the whole period of his service with the Board, when not holding the office of President, he was at the head of one of its important Standing Committees.

It was the time of the greatest improvement and development of the public school system in St. Louis, when it was brought from an exceedingly low ebb of inefficiency to a high tide of successful administration, taking its place easily as the best in the United States and the model for the work of other cities. It is not too much to say that this attainment was largely the result of Mr. Moore's quiet courage and intelligent labors.

Mr. Moore took a most active part in the ethical movement in St. Louis. He became Chairman of the Ethical Society in 1891, and, thereafter, until disabled by ill health, he remained as its active civic representative and its guiding spirit as Chairman Emeritus until his death. He worked for his creed with an unselfishness, a devotion, and a persistency worthy of the best traditions of his Puritan ancestors.

Mr. Moore's most marked characteristic was perfect mental integrity. He deceived no man, and, better still, he did not deceive himself. He cared nothing for praise or blame from the public, if he thought he was right. He had no ambition for himself. He was quiet, dignified, and reserved, a man of few words, with a marked sense of fairness and justice. This trait was so generally recognized that in all the work coming under his charge, covering a

period of a great many years, he never had any disputes to litigate. His letters, papers, and addresses were models of clear thinking and concise diction.

During his long and active career, he was associated with, and had in his employ, many young men entering the Profession, whose success is due in a large measure to the inspiration of his example, and whose association with him will always remain among their most cherished recollections.

Mr. Moore was married on October 3d, 1878, to Alice Filley, of St. Louis, who, with a son, Charles Whippo Moore, survives him.

He was a Member of the Institution of Civil Engineers of Great Britain and one of the early members and a Past-President of the Engineers' Club of St. Louis.

Mr. Moore was elected a Member of the American Society of Civil Engineers on April 5th, 1876. He served as a Director of the Society from 1892 to 1893, as Vice-President in 1888, and from 1889 to 1900, and as President in 1902.

"They cannot die whose lives are part
Of that great life which is to be,
Whose hearts beat with the world's great heart,
And throb with its high destiny.

"Then mourn not those who, dying, gave
A gift of greater life to man;
Death stands abashed before the brave;
They own a life he may not ban."

IGNACIO MAREA de VARONA, M. Am. Soc. C. E.*

DIED MAY 12TH, 1922.

Ignacio Marea de Varona, the son of Ignacio M. and Angela M. (de Aguero) de Varona, was born at Puerto Principe, Cuba, on February 4th, 1842. His ancestors were members of prominent Spanish families in Cuba. He received private instruction previous to entering Mr. Berger's French School, in New York City, where he was prepared for the Rensselaer Polytechnic Institute, Troy, N. Y. He entered the latter institution in September, 1860, and was graduated therefrom in 1863, with the degree of C. E.

Mr. de Varona began his professional work immediately after his graduation, and continued the practice of his profession throughout his long life. The earlier part of his career was spent on railroad work, in Cuba, South America, Europe, and the United States. From 1864 to 1869, he was Resident, Assistant, and Chief Engineer of the Nuevitas and Puerto Principe Railroad. As Chief Engineer, he designed and built the extension of that railroad to, and terminal works in, the Bay of Nuevitas. Later, from 1878 to 1881, he was again Chief Engineer of this railroad, and reconstructed important parts thereof.

* Memoir prepared by William W. Brush and Merritt H. Smith, Members, Am. Soc. C. E.

In 1882, Mr. de Varona joined the Engineering Corps of the City Works Department of Brooklyn, N. Y., and was assigned to water supply work. This was the beginning of his connection with the development of the water supply of New York City, on which he was engaged until his retirement in 1914, after a service of more than thirty-two years. During the Eighties, he was assigned, first as Assistant Engineer, and, later, as Division Engineer, in connection with the extension of the water supply system of Brooklyn, east of Rockville Centre, N. Y. On the completion of this work, he was appointed, on July 1st, 1894, by the then Commissioner of Public Works, Mr. Alfred T. White, as Engineer of Water Supply, having charge of all the work connected with the extension and improvement of the water supply system, exclusive of the distribution system.

On the creation of Greater New York, in 1898, Mr. de Varona was continued as Engineer of Water Supply of the Brooklyn System, and, in 1902, he was appointed Chief Engineer of the Bureau of Water Supply, in charge of the Boroughs of Brooklyn and Queens, the distribution system at this time being also placed under his jurisdiction. He remained in charge of the Borough of Queens for only a short time, but continued as Chief Engineer in charge of Brooklyn until 1905, when he was appointed Chief Engineer with jurisdiction over all the Boroughs of Greater New York. In 1906, a separate Acting Chief Engineer was appointed for the Borough of Brooklyn, Mr. de Varona continuing in charge of the other Boroughs. In 1910, the organization of the Department was remodeled, and Mr. de Varona was appointed Chief Engineer of the Bureau of Water Supply, having charge of the maintenance, operation, and extension of the water supply system for the entire city, with the exception of the construction of the new Catskill System, which was under the jurisdiction of the Board of Water Supply, an entirely separate city organization. He remained in charge of the Bureau of Water Supply until July 1st, 1914, when he retired on a pension. After his retirement, he continued the practice of his profession, as a Consulting Engineer, but was not very actively engaged on engineering work.

The important professional work of Mr. de Varona was the management and improvement of, first, the water supply system of Brooklyn, and, later, that of Greater New York. He was a man of unusual mental development and physical and nervous energy. As an engineering executive, he would quickly sense the strong and the weak points of any plan presented to him, or of any proposed line of action, and he energetically fought for what he thought represented the proper course. This was well exemplified in the proposed filtration of the Croton supply. In 1911-12, plans were prepared, under Mr. de Varona's direction, for a slow sand filtration plant, to be located on the site of the Jerome Park East Basin Reservoir, the total estimated cost of this work having been about $8 000 000. After the bids for the main contract had been received, opposition to the work developed, the necessity of the purification of the Croton supply by filtration being questioned. No criticism was made as to the adequacy of the plans to accomplish the purpose sought, and Mr. de Varona strenuously fought for the filtration project, but the Board of Estimate ultimately decided not to proceed with this work.

The most important developments of the water supply system of New York City conducted under his direct charge, were: The utilization of the underground water supply of Long Island by wells and gallery system; the design and construction of the high-pressure fire service systems in Brooklyn and in Manhattan; and the remodeling of the distribution system in Brooklyn. Mr. de Varona devoted virtually his entire time to the water supply system of New York City, occasionally, however, he aided other communities as Consulting Engineer, with advice regarding water supply and sewerage problems.

Although interested in the work of the Engineering Societies, Mr. de Varona did not frequently attend their sessions. Outside his official duties, his interest was especially in his home, where he died on May 12th, 1922. He is survived by several sons and daughters.

Mr. de Varona was elected a Member of the American Society of Civil Engineers on April 7th, 1886.

WILBUR FISK FOSTER, M. Am. Soc. C. E.*

DIED MARCH 26TH, 1922

Wilbur Fisk Foster was born at Springfield, Mass., on April 13th, 1834, being the third of six children. His father, Dexter Foster, of Ashfield, Mass., was a lineal descendant in the fifth generation of Peregrine White who was born on the *Mayflower,* in November, 1620. His mother was Euphrasia Maria Allin, of Enfield, Conn.

After some years of preparatory studies, Dexter Foster began work in 1838 as Civil Engineer on the surveys, location, and construction of the Western Railroad of Massachusetts, from Springfield, Mass., westwardly to the New York State line, and of the Albany and West Stonebridge Railroad from the State line to Albany, N. Y. On the departure of Maj. George W. Whistler, the first Chief Engineer of the Western Railroad of Massachusetts, for Russia as Chief Engineer of the St. Petersburg and Moscow Railroad, Capt. John Childe became Chief Engineer, with Dexter Foster as his First Assistant, and he served as such until the completion of the road to Albany, N. Y., on April 22d, 1843. Among the incidents of this service was the location and construction of the first railroad tunnel on a curve alignment. While engaged under Capt. Childe, on the survey of the Connecticut River Railroad, north from Springfield, Mass., Dexter Foster died on August 11th, 1844.

The life of Wilbur Fisk Foster during the next seven years was a lively skirmish for his own support and such meager help as he could give his mother. His education was received in the public schools of Northampton and Springfield, Mass., and he was graduated from the High School at Springfield in 1851, with preparation for entry as a Sophomore at Yale University, but this was given up for lack of funds.

* Memoir prepared by J. S. Walker, M. Am. Soc. C. E.

Capt. Childe who was then Chief Engineer of the Mobile and Ohio Railroad, gave him employment on that road. Mr. Foster left Springfield, on December 29th, 1851, traveling by railroad to Cumberland, Md., thence by stage to Wheeling, Va., by steamboat to Paducah, Ky., and thence by steamboat to Hamburg, Tenn., near which place he joined the late Adna Anderson, M. Am. Soc. C. E., and a locating party, as Rodman. He was engaged on the location of the line to the mouth of the Ohio River and on preliminary branch lines to Columbus, Paducah, and Hickman, Ky., and Mt. Pleasant, Tenn. On the completion of the maps and profiles, the party was dispersed at Columbus, Ky. Mr. Foster went, as ordered by Capt. Childe, in company with Mr. Anderson, to Mobile, Ala., where he reported to Maj. John W. Goodwin, Resident Engineer. He was assigned as Instrumentman on the construction of the division from Boice to Quitman, Miss., in charge of the late Horace Harding, M. Am. Soc. C. E., Division Engineer.

On April 12th, 1853, on order of Capt. Childe, Mr. Foster was relieved from duty on the Mobile and Ohio Railroad, with instructions to report to Mr. Anderson at Columbia, Tenn., for service on the Tennessee and Alabama Railroad, of which Capt. Childe was Chief Engineer and Mr. Anderson, Resident Engineer. From this time until April 25th, 1861, Mr. Foster was continuously engaged under Mr. Anderson who was Chief Engineer of the Central Southern Railroad, from Columbia, Tenn., south to the Alabama line, the Edgefield and Kentucky Railroad, and the Hendersonville and Nashville Railroad, which, together with the Tennessee and Alabama Railroad, when built, and after several consolidations, were finally absorbed into the Louisville and Nashville Railroad, constituting its line from the Alabama State line, at Veto, to Henderson, Ky.

Mr. Foster began work as Transitman on April 21st, 1853, on the preliminary survey from Columbia, Tenn., toward Florence, Ala., and Hamburg, Tenn. On the completion of these surveys, he was assigned to duty as Instrumentman on the construction of the Second Division of the Tennessee and Alabama Railroad, from Spencer's Creek, north of Franklin, Tenn., to Spring Hill, under Mr. J. H. Devereux, Division Engineer, who afterward became a prominent and well known railroad executive. From July 24th to November 6th, 1854, Mr. Foster had charge of track-laying from Nashville south, and, again, from June to November, 1855, he had charge of track-laying from Franklin south.

From November 5th, 1854, to January 15th, 1855, he was employed as Transitman on surveys of the Edgefield and Kentucky Railroad, from Nashville to the Kentucky line, in the direction of Trenton, Ky., and from January 15th to April, 1855, he served in the same capacity on the surveys of the Central Southern Railroad from Columbia, *via* Pulaski, to the Alabama State line, in the direction of Athens, Ala. The next few months were spent by Mr. Foster on a visit to his home in New England.

On the completion of his work on track-laying south of Franklin, Ky., he was engaged, on November 20th, 1855, as Transitman on the surveys and final location of the Edgefield and Kentucky Railroad, from Edgefield Junction, Tenn., to Guthrie, Ky. During much of the time, he acted as Chief of Party.

On the completion of the location and the usual plans and estimates, he was assigned to duty at the office in Nashville for the several railroad companies of which Mr. Anderson was Chief Engineer.

The first railroad bridge across the Cumberland River was that constructed at Nashville for the joint use of the Louisville and Nashville and the Edgefield and Kentucky Railroads. It was built, under the charter of the latter road, under the direction of Mr. Anderson, Chief Engineer. In September, 1856, Mr. Foster was assigned as Engineer in charge of this work and made all the surveys, soundings, location of piers, and abutments, prepared all the plans, and supervised every detail until the completion of the masonry ready to receive the superstructure, which, combined with special duties from time to time on the different lines of the Company, was his work for two years. The masonry of the Cumberland River Bridge having been practically completed on August 1st, 1858, he was given charge of the construction of the Third Division of the Central Southern Railroad from Pulaski to the Alabama State line, at the present station, Veto, of the Louisville and Nashville Railroad. This construction included bridges over Richland Creek and Elk River, and a tunnel 1 230 ft. long, through Madry Hill, also much heavy rock grading and trestle work. On the completion of the tracklaying, he finished the work to a connection with the South and North Railroad at the Alabama State line on November 24th, 1860.

From December 15th, 1860, to April 25th, 1861, Mr. Foster was employed on the surveys and location of the Hendersonville and Nashville Railroad, from Hopkinsville to Sebree, Ky., and in retracing the line and levels of the part already graded, from Guthrie to Hopkinsville and from Sebree to Henderson, practically completing the field work, maps, profiles, and estimates.

From the foregoing record, it is evident that Mr. Foster's work from the time of his arrival at Hamburg, Tenn., in 1852, was education in the highest degree, and his training therein had made him a competent civil engineer of experience.

He enlisted as a private in Company C (Rock City Guards), on April 25th, 1861, and was mustered into service as a private in the First Tennessee Regiment, thus becoming a Confederate soldier. He was detailed on engineer duty in making surveys of the Tennessee and Cumberland Rivers near the Kentucky line, and had charge of construction at Fort Donelson, where he designed the water battery. Afterward, reporting to Col. Bushrod Johnson, Chief Engineer Officer, he was placed in charge of the construction of Fort Henry, although he remonstrated against its location. Mr. Foster was commissioned a First Lieutenant of Engineers on September 14th, 1861, and ordered to Cumberland Gap, where he built earthworks, batteries, and block houses, and made a topographical map of the vicinity. When Gen. Kirby Smith advanced into Kentucky, Lieut. Foster was promoted to be Captain of Engineers, and was, successively, on the staffs of Generals Bragg and Joseph E. Johnston, in charge of topographical work. He was promoted to be Major of Engineers on March 17th, 1864, and on August 7th, 1864, he was assigned to the staff of Gen. A. P. Stewart, serving in that position until Johnson's surrender at Greensboro, N. C. In the performace of his duties, Maj. Foster was under fire in many

battles, the most important of which were Chickamauga, Dalton, Resaca, New Hope Church, Kennesaw Mountain, Atlanta, Jonesboro, Franklin, and Nashville.

It has been stated that, "he was trusted implicitly by General Albert Sidney Johnston and General Joseph E. Johnston, and he was active in the Atlanta campaign under the latter, receiving high commendation for his work in map making and for the intelligence displayed in laying out the Confederate works, rifle-pits, breast-works, etc."

After the surrender, Maj. Foster returned with his detachment to Macon, Ga., where they separated, and he made his way to Nashville, walking part of the way. He reached Nashville on May 15th, 1865, after four years' service in the Army of the Confederacy, which he stated was "an experience I shall never forget or regret." After a few days, he went to visit his mother in New England. He returned to Nashville on July 17th, 1865, and shortly afterward opened an office for the general practice of engineering, which he maintained until 1884, when it is was merged with a contracting business, which soon became the principal feature. On July 15th, 1865, the Broad Street Bridge Company appointed him as Engineer to supervise the reconstruction of the Suspension Bridge across the Cumberland River, at Nashville, which had been destroyed during the war. This work was completed, and the bridge was opened for traffic in August, 1866. On July 26th, 1865, he was also appointed Engineer on the construction of the first street railway built in Nashville, and remained in charge of this work until its completion in 1866.

On January 1st, 1866, Maj. Foster was made Resident Engineer of the Edgefield and Kentucky Railroad (now part of the Louisville and Nashville Railroad), and continued in that duty until November 15th, 1869, during which period most of the bridges and trestles were rebuilt and the roadbed and track placed in good condition. From early in 1866 until July 1st, 1872, when the Nashville and Decatur Railroad was leased to the Louisville and Nashville Railroad Company, he was constantly in the employ of J. W. Sloss, President of the Nashville and Decatur Railroad Company, as Engineer of the Company, and made frequent trips of inspection. Plans for stone arch culverts were prepared by him and were built under his direction. For many years, beginning in 1867, he was frequently employed at times by the Nashville and Cincinnati Railroad Company in the location of its real estate in Nashville and along the line; he also made an inspection and report on the condition of the bridges and trestles between Nashville and Hickman, Ky., in October, 1869.

From 1871 to 1876, Maj. Foster was entrusted with much engineering service for the Louisville and Nashville Railroad Company, about the most important being the location, acquisition of land, and plans for constructing a connecting line between that road and the Nashville and Decatur Railroad, in Nashville.

By the appointment of Mr. Sloss, President of the Nashville and Decatur Railroad Company, he made a preliminary survey in December, 1867, and furnished map profiles and estimates of cost of a proposed branch railroad

from Culleoka to Lewisburg. In 1868, he made the necessary surveys and prepared a map of Davidson County, Tennessee, for the County Commissioners, at a cost of $3 106.

In 1869, he made a survey map, profiles, and estimates of quantities for a proposed railroad from Murfreesboro to Lebanon, Tenn., as well as preliminary surveys, map profiles, and estimates of quantities for a proposed railroad from Lebanon, *via* Hartsville, Tenn., to the State line in the direction of Scottsville, Ky. He also made a study of the State Fair grounds, cross-sectioned it, and designed and built a 1-mile trotting track, a grand stand, and a spur-track to the Nashville, Cincinnati, and St. Louis Railway. This is the same ground on which the Tennessee Centennial was held in 1897, and is embraced in the present Centennial Park. Laying out, grading, and macadamizing West End Avenue, from 15th Avenue to Cockrills Springs, and the construction of the Nashville and Fair Grounds Street Railroad from the city limits to the Fair Grounds, was done by contract of the property owners, with Maj. Foster as Engineer and Contractor, the work being completed early in October, 1871.

The improvement of the Capitol grounds was undertaken by the Commissioners of the Penitentiary, in accordance with plans furnished by the late John Bogart, M. Am. Soc. C. E. Maj. Foster made a topographical map of the site and furnished a copy to Mr. Bogart who submitted designs. Maj. Foster took charge of the construction, developed the general and special designs, furnished all the detailed drawings of the cut-stone work, and had general supervision of the work in every detail. This work was done by convicts between July 12th, 1871, and May 31st, 1876, and no more pleasing retaining wall can be found anywhere.

On October 12th, 1865, Maj. Foster was elected City Engineer of Nashville, and with the exception of the period from October 12th, 1867, to July 1st, 1869, he held the office by re-election or appointment until March 8th, 1884, when he "peremptorily resigned". There were no records for guidance as to sewers, water pipe, street grades, no profiles, no field books, no record whatever of street alignment, or other details. There had been no control over improvements made by property owners, every one doing as it pleased him in the matter of grades, sidewalks, etc. When he resigned the office of City Engineer on March 8th, 1884, Maj. Foster left the Department completely organized and turned over to his successor complete records of more than three hundred streets, alleys, and sewers, as well as a thoroughly tested system of bench-marks and references covering the entire city at that date. The total cost to the city of the Engineering Department for the 16$\frac{2}{3}$ years of Maj. Foster's service, including every item of expense for salary, assistants, office rent, office expenses, etc., was less than $40 000. The work done during this period reflects the highest credit on Maj. Foster as a skilled engineer, possessed of high ideals and character, and a devoted public spirit. The City of Nashville was truly fortunate in having him for City Engineer during the time he served as such.

The North Nashville Bridge across the Cumberland River, near Hyde's Ferry, was built by the County of Davidson, represented by a Board of Com-

missioners, and under plans and specifications prepared by the late E. F. Falconnet, M. Am. Soc. C. E. The contracts for the various details were made by the Commissioners under the advice of Maj. Falconnet, and the work was begun and rapidly pushed under his direction. After his death on October 17th, 1887, Mr. H. C. Gardner and O. H. Landreth, M. Am. Soc. C. E., were, successively, in charge. On February 28th, 1888, Maj. Foster was appointed Engineer on the part of the Commissioners, and he directed the completion of the substructure and the inspection and erection of the superstructure until its completion and acceptance on April 1st, 1889.

In April and May, 1891, Maj. Foster made surveys and prepared plans, profiles, and estimates for a boulevard, 80 ft. wide, from the Belmont School grounds southwardly 2¾ miles, to an intersection with the Granny White Pike. Later, in 1893 and 1905, he made a subdivision of the Belmont Land Company north and east of this Boulevard. Of this work, Maj. Foster said: "This was my last specific engineering engagement; such other engineering service as rendered by me after 1884 being of little importance or incidental to my business as contractor."

The firm of Foster and Creighton was a partnership formed in 1885 by Maj. Foster and Mr. Robert C. Creighton, which was engaged in the contracting business. In 1907, Maj. Foster sold his interest in the firm and retired from business, and the Foster and Creighton Company was incorporated. Maj. Foster served as a Director in the Company for about five years, until 1912. The Company did a large amount of work, all of which was executed in a thorough, workmanlike manner, and was satisfactory to its patrons.

After Maj. Foster retired from business, he spent his time mostly in reading and writing. Most of his writing pertained to Masonry, but he prepared his engineering record, which furnished the data given herein. His war record was taken from the "Confederate Veteran" and "Confederate War History".

Maj. Foster was one of the charter members of the Engineering Association of the South, which was organized in 1889, and served as its President in 1899. The successor of this Association, the Nashville Engineering Association, made him an Honorary Member. He was also a Charter Member of the Hermitage Club and of the University Club, both of Nashville. At one time, he was a Director of the Phillips and Buttorff Manufacturing Company, and, at the time of his death, he was a Director of the Nashville Trust Company, which, on March 30th, 1922, passed the following resolution:

"It is with profound sorrow we record the death of our dear friend and associate, Major W. F. Foster, who at 11:50 in the evening of March 26th, 'passed from the land of shadow into the land of light'.

"Major Foster had served this Company on the Board of Directors, faithfully and efficiently, for twenty-five years.

"He was a man unusually endowed, of marked individuality, lofty ideals, broad sympathies, and unswerving integrity; an accomplished gentleman, a skilled civil engineer, a successful business man, an ardent Christian and churchman, and a distinguished Mason. His long life of eighty-eight years was a gospel of good will and good works, spent in unselfish devotion to God, his family and his fellow man. His influence was gentle, kindly and far-reach-

ing. His presence always gave an added charm and a sweet old-fashioned dignity to any circle in which he moved.

"The memory of this good man is enshrined in our hearts, and we extend to his dear wife and loved ones, our deepest sympathy and commend them to Him 'who is able to ransom from the power of the grave and who turneth the shadow of death into the light of the morning'.

" 'When death draws down the curtain of the night, and those we love in his deep darkness hides, we can but wait the coming of the light, and bless the memory that still abides.'

"Therefore, Be It Resolved, That this expression of our respect and remembrance be inscribed on a memorial page of the Minute Book of this Board, and a copy be sent the family."

Maj. Foster joined the First Presbyterian Church of Nashville on September 8th, 1868. He was elected a Deacon in 1873 and an Elder in 1896. He was constant in attendance and a devoted member, performing all his duties with credit to himself and his church for more than fifty years. On his death, the pastor and officers of the First Presbyterian Church adopted the following resolution:

"On March 26th, 1922, after a life of nearly four score and ten years, Major Wilbur Fisk Foster, having finished his course and kept the faith, was called from the church militant to the church triumphant, 'and entered into the rest prepared for the people of God.'

"Major Foster was a man of the strictest integrity and his character was unimpeached and unimpeachable. No one who knew him intimately could fail to admire his gentle and affable nature and to recognize in him a true man.

"In the death of Major Foster the city has lost a most valuable citizen, and one who has left his impress on it by his work as an official in past years, and his godly walk before the people.

"For nearly forty years he was an officer in this church and, until his physical affliction, in the loss of his hearing, prevented him from attending the meetings of the session, he was usually present, and his opinion and suggestions in regard to matters pertaining to the good of our church were always regarded as safe and sound.

"While we bow with submission to the will of our Heavenly Father, yet we cannot but mourn the great loss our church and official board have sustained in the death of Major Foster. We shall enshrine his memory in our hearts, and count ourselves happy to have had such a friend.

"Resolved, That a copy of these resolutions be sent to the family to whom we, as his Pastor and fellow officers, extend our sincere sympathy in their irreparable loss."

From the time he arrived at manhood, Maj. Foster gave so much of his time and attention to the Masonic order and its various branches, that his Masonic activities may be said to have formed a large and important part of his career. It is deemed appropriate, therefore, to set out briefly herein a record of his Masonic career, as authenticated by Mr. Garnett N. Morgan, Sovereign Grand Inspector General in the State of Tennessee, of the Supreme Council, Ancient and Accepted Scottish Rite, who states:

"It is very difficult to give in a very few words an adequate idea of the splendid Masonic career of Major Wilbur Fisk Foster 33°.

"His services to Freemasonry in Tennessee were brilliant in character and the honors of the Fraternity were showered upon him. He served not only as the Worshipful Master of the two Blue Lodges of which he was a member during his long and useful life, but became the presiding officer of each of the

other Masonic Bodies in which he held his membership—Royal Arch Chapter, Council of Royal and Select Masters, and Commandery of Knights Templar. He was elected the presiding officer of the Grand Bodies of the State—Grand Lodge, Grand Chapter, Grand Council, and Grand Commandery—and served with distinction in each of these exalted stations. He also rendered eminent services to the General Grand Chapter of the United States.

"In the Ancient and Accepted Scottish Rite of Freemasonry, he assisted in organizing the Scottish Rite Bodies of the Valley of Nashville and for a number of years was Master of Kadosh of Trinity Consistory. In recognition of his services to the Rite, he was honored by the Supreme Council with the Thirty-third Degree.

"Major Foster was one of the most influential of Tennessee Masons, a Masonic scholar and author of note, and one of the ablest Masonic ritualists the State has produced."

Maj. Foster was married on June 19th, 1866, to Miss Elizabeth Nichol, daughter of John Nichol, of Nashville, a member of one of the old and respected families of Tennessee. His widow, a daughter, Mrs. C. C. Foster, and two granddaughters, survive him.

Maj. Foster was endowed with unusual common sense, discretion, and a conscience. He was a modest, reticent, honest man, devoted to his family, his church and his Masonic connections, a gentleman of the old school, loyal to all interests confided to him, a thoroughly trained civil engineer, a brave, efficient Engineer Officer of the Confederate Army, methodical, prompt, and accurate. He enjoyed good health, and maintained all his faculties to the last, except that he was afflicted with deafness for several years before his death. He walked from his home to his business, and return, almost daily, until a few days before his death.

Major Foster was elected a Member of the American Society of Civil Engineers on May 7th, 1873, and only a few living members of the Society have had a longer connection with it. Major Foster began his engineering work, the same year the Society was instituted.

ROBERT CAMPBELL GEMMELL, M. Am. Soc. C. E.*

DIED OCTOBER 25TH, 1922.

Robert Campbell Gemmell, the son of Robert Brown Gemmell and Anna Eliza Campbell Gemmell, was born at Port Mathilda, Pa., on July 5th, 1863. In the early Seventies, his parents moved to Topeka, Kans., where Mr. Gemmell secured a common school and high school education. Choosing Engineering as his profession, he entered on its study at the University of Michigan, from which institution he received his degree of B. S., in 1884. In 1895, his Alma Mater conferred on him the degree of C. E., and, in June, 1913, in recognition of his valuable services as an Engineer, honored him with the degree of Master of Engineers.

Following his graduation, Mr. Gemmell entered the Engineering Department of the Atchison, Topeka and Santa Fé Railway Company, and became a

* Memoir prepared by L. S. Cates, Gen. Mgr., Utah Copper Co., Salt Lake City, Utah.

Division Engineer thereof, in charge of construction work. In 1891, he located at Pendleton, Ore., and engaged in private practice, which extended over Oregon, Washington, and Idaho. In 1893, he moved to Salt Lake City, Utah, where, continuing his private practice, he was also employed in the Engineering Department of that city.

In the course of his practice in Salt Lake City, Mr. Gemmell was made Engineer for the mines of Capt. Joseph R. DeLamar, including the Golden Gate and the Consolidated Mercur properties at Mercur, Utah. He was the first State Engineer of Utah, holding that position from 1898 to 1901, when he went to Mexico as Manager of the Mexican Syndicate Mining Company, to become later Superintendent of the Mexican properties of the Guggenheim Exploration Company. Journeying to Spain in 1905, to examine some gold mining properties for a group of New York capitalists, he returned to Salt Lake City in January, 1906, and immediately became associated, in the capacity of Superintendent of Mine and Mills, with the Utah Copper Company, the banner copper mine of the world.

In 1909, Mr. Gemmell was appointed Assistant General Manager of the Utah Copper Company, and, on May 1st, 1913, was made General Manager thereof. He was also consulting Engineer for the Ray Consolidated Copper Company, the Chino Copper Company, and the Nevada Consolidated Copper Company, as well as General Manager of the Bingham and Garfield Railway Company. In June, 1919, he was advanced to the position of Assistant Managing Director of the Utah, Ray, Chino, and Nevada properties of the Utah Copper Company.

He is survived by his widow, Belle Evans Anderson Gemmell, to whom he was married, at Salt Lake City, Utah, on October 17th, 1888.

A lover of Nature, Mr. Gemmell was fond of outdoor sports, particularly of hunting, fishing, and golf. He freely gave of his life and means to all that promised betterment of American boyhood. Although his chosen profession was his life work, he devoted much of his time and talents to public affairs, and to numerous business enterprises wherein his conservative judgment and business sagacity were invited and appreciated. Gracious in manner, gentle in spirit, manly in everything, his personality, not only commanded the admiration, but easily and quickly won the affection, of his associates, and gave him an acknowledged power among men. He was a true man whom it was a rare privilege to know and an honor to call friend.

As an engineer, Mr. Gemmell rose to eminence both in ideals and achievement. He built to the level of his best opportunities, because he built down to the solid foundation of a sense of obligation. Capable, thorough, skilled, and accomplished, he was patient and chivalrously considerate of those under his industrial authority to a surpassing degree, yet never satisfied with anything short of economic perfection. In his death, the Society has lost one of its most valued members, and the Profession one of its most able exponents.

Mr. Gemmell was a Member of the American Institute of Mining and Metallurgical Engineers, the Mining and Metallurgical Society of America, and the Utah Chapter of the American Mining Congress (of which Local Chapter he was the prime organizer and first Governor). He was also a Member of the

Sons of the American Revolution, and Honorary President and National Representative of the Salt Lake Council of the Boy Scouts of America. He was appointed by the Governor of Utah on the Committee which, in 1916, formulated the Workmen's Compensation Act of that State. He was a member of the Alta, University, Country, Commercial, and Rotary Clubs of Salt Lake City, of the Rudy Duck and the Bear River Duck Clubs, of Utah, the Flat Rock Club, of Idaho, and the Sequoyah Country Club, of Oakland, Calif.

Mr. Gemmell was elected an Associate Member of the American Society of Civil Engineers on October 5th, 1892, and a Member on December 4th, 1895.

MAURICE STILES PARKER, M. Am. Soc. C. E.*

DIED JUNE 3D, 1921.

Maurice Stiles Parker was born at Groveland, Mass., on December 3d, 1851. He received his education at the West Point Military Academy and the University of Berlin, which he entered in 1871, and where he studied civil and mechanical engineering for three years. Before leaving Europe, he visited many of the important engineering works on the Continent.

Returning to the United States, in the spring of 1875, Mr. Parker established an office at La Crosse, Wis., and carried on a private engineering practice until July, 1877. From July, 1877, to July, 1881, he was Chief Clerk of the Mining Department of Dakota Territory, in the office of the United States Surveyor-General, and from July, 1881, to January, 1882, he was United States Deputy Mineral Surveyor, locating flumes in the Black Hills, Dakota Territory, and Consulting Engineer to Samuel McMaster, Superintendent of the Homestake Mine.

From April, 1882, to June, 1883, Mr. Parker served as Division Engineer in Charge of Construction on the Chicago and Northwestern Railway, and from August, 1883, to April, 1884, he was engaged as Locating Engineer for the Chicago, Burlington, and Northern Railway Company.

He held the position of Acting City Engineer of Winona, Minn., in charge of public improvements, from April, 1884, to April, 1885, and from April, 1885, to March, 1886, he was Resident Engineer of the Chicago and Northwestern Railway. He was employed as Locating Engineer of the Montana Extension of the St. Paul, Minneapolis, and Manitoba Railway from April, 1886, to January, 1888, when he opened an office as Consulting Engineer, at Winona, Minn., serving also as Chief Engineer of the Sioux Falls, De Smet, and Northern Railway.

In 1890, he was appointed Chief Engineer of the Great Falls Water Power and Townsite Company and had charge of the construction of the Black Eagle Dam, at Great Falls, Mont., in 1890-91, which was the first power development on the Missouri River. Later, he became engaged in railroad contracting in Montana and afterward in the States of the Southwest. For a

* Memoir compiled from information on file at the Headquarters of the Society.

number of years before his death, which occurred on June 3d, 1921, at Spokane, Wash., Mr. Parker had made his home at St. Maries, Idaho, where he was engaged in water-power development and drainage projects in Montana and Northern Idaho.

He was married in 1875 to Miss Minneowa Burns, in Geneva, Switzerland, who died in 1917. Mr. Parker is survived by his daughter, Mrs. Charles W. Craney, of St. Maries, Idaho.

Mr. Parker was elected a Member of the American Society of Civil Engineers on February 5th, 1890.

SAMUEL EVERETT TINKHAM, M. Am. Soc. C. E.*

DIED APRIL 21ST, 1921.

Samuel Everett Tinkham, the son of Samuel Miller and Celia (Ellis) Tinkham, was born at Taunton, Mass., on March 31st, 1852. He descended from a long line of New England ancestors some of whom came to New England in the *Mayflower*. His father was Treasurer of the Taunton Foundry and Machine Company, and afterward established a music store in Taunton, which he conducted for many years. Mr. Tinkham's mother died when he was only three years of age. His father afterward married Miss Margaret E. Bright, who, having been a teacher, was well qualified to direct the education of her four stepsons.

It was through her efforts, that Mr. Tinkham was able to qualify for the Massachusetts Institute of Technology and to become a member of the Class of 1873. During the summer of 1872, he was engaged in active field work as Rodman in the Engineering Department of the New York State Canals. The summer following his graduation, he was appointed an Assistant in the Corps of Engineers, United States Army, and for more than a year was employed on harbor improvements at Edgartown, Mass.

In October, 1874, Mr. Tinkham was appointed as Transitman in the office of the City Engineer of Boston, Mass. In addition to his regular work, he made experiments with the blue-print process, then in its infancy, and thus helped to develop and to make the process of reproduction by blue-printing what it is to-day.

During the first eight years of his service with the City of Boston, from 1874 to 1882, Mr. Tinkham was engaged principally on bridge work, under the late John E. Cheney, M. Am. Soc. C. E., who was then City Bridge Engineer. At this time, several city bridges were built or reconstructed, among which were the Dover Street, Meridian Street, Broadway, and Chelsea Bridges. Part of Mr. Tinkham's time was also given to work in connection with the Boston Main Drainage System, especially with the design and construction of the Calf Pasture Pumping Station.

In the fall of 1882, Mr. Tinkham left the employ of the City Engineering Department with the idea that he could find a more promising future else-

* Memoir prepared from information on file at the Headquarters of the Society.

where. He became Assistant Engineer on the New York and New England Railroad, in charge of the design and erection of bridges for the double-tracking of that line. His service with this road was terminated in April, 1884, when he returned to Boston and again entered the office of the City Engineer, where he remained for thirty-seven years, until his death.

On his return to municipal work, Mr. Tinkham was given the title of Assistant Engineer and Principal Draftsman of the Engineering Department. He became Chief Assistant to the Bridge Engineer and his work was largely in charge of bridge designing. Among the bridges constructed at this time were the Allston, Beacon Street, Boylston Street, Brookline Avenue, and Cottage Farm Bridges, over the Boston and Albany Railroad; the Byron Street Bridge, in East Boston; the Dover Street and L Street Bridges, in South Boston; the draw-span of the Neponset Bridge; and the Harvard Bridge across the Charles River, which was built jointly by the Cities of Boston and Cambridge, under the direction of the Boston Engineering Department. In addition to bridge work, Mr. Tinkham was also engaged on the Boston Improved Sewerage System, the Mystic Pumping Station, and on work in connection with the elimination of grade crossings.

During the late Nineties, several new bridges were built, the most important of which were the Charlestown and Cambridge Bridges. The South Terminal Station which was to unite the railroads on the south side of the city, was also projected about this time and involved much planning, including the rearrangement of the streets and extensive constructions along the water-front of Fort Point Channel. Mr. Tinkham was directly in charge of the municipal work in this connection.

He was engaged, during the last twenty-three years of his service with the City Engineering Department, principally on construction engineering, which included bridges, sea walls and retaining walls, tide-water foundations, harbor shore protection, ferry improvements, wharves, and other water-front structures.

Mr. Tinkham was also connected with other well-known projects, among which were the Northern Avenue and Sleeper Street improvement, from Atlantic Avenue, across Fort Point Channel and the South Boston Flats, involving the construction of one of the most important of the City's tide-water bridges; grade-crossing elimination, including the building of a number of new bridges over railroads; the rebuilding of the Chelsea Bridge north over the Mystic River, including the heaviest swing draw-span in New England; the building of the new swing draw-span of the Broadway Bridge, over Fort Point Channel; the rebuilding of the Meridian Street and Chelsea Street Bridges, between East Boston and Chelsea, each with a swing draw-span; the reconstruction of the Malden Bridge between Charlestown and Everett; and the rebuilding of the Granite Avenue Bridge over the Neponset River.

In 1911, the City Engineering Department, the Street Department, and the Water Department, of Boston, were united to form the present Public Works Department, and Mr. Tinkham was made Construction Engineer of the Bridge and Ferry Division.

In addition to his work for the city, he was frequently called on as Consulting Engineer. In 1897, he was appointed as one of the members of the first Board of Examiners for Civil Engineers in the classified service, a position which he held for more than fifteen years, until January, 1913.

Mr. Tinkham was primarily a Bridge Engineer, but he was also an authority on foundation problems, particularly on constructions in tide-water. He was absolutely honest, of a cheerful disposition, and was an agreeable companion. He died at his home in Roxbury, Mass., on April 21st, 1921, of apoplexy.

Mr. Tinkham was married on October 20th, 1879, to Miss Louisa W. Danforth, of Roxbury, who, with three children, Charles S. Tinkham, Resident Engineer of the Massachusetts Highway Commission at Greenfield, Mass., Frank B. Tinkham, of Boston, and Mrs. Leo Twombly, of East Orange, N. J., survives him.

He was Secretary of the Boston Society of Civil Engineers for thirty-six years, and his efforts in behalf of the interests of that Society were indefatigable and highly appreciated by every member. Its successful history and rapid development were due largely to his untiring and self-sacrificing interest. He was a member of the New England Water Works Association and the Massachusetts Highway Association.

Mr. Tinkham was elected a Member of the American Society of Civil Engineers on March 2d, 1892.

LUTHER WAGONER, M. Am. Soc. C. E.*

DIED JULY 1ST, 1922.

Luther Wagoner was a remarkable man. He represented the self-made engineer who had reached the highest standard in theoretical work by reason of intense application and studious habits. Those of his friends who knew him best, admired the facility with which he could recall events and reproduce from memory the details of the technical problems and papers which he had had occasion to study.

Luther Wagoner was born at Connersville, Ind., on May 23d, 1847. When he was only 19 years of age, he was appointed Deputy County Surveyor of Johnson County, Missouri. He was engaged on railroad work in 1868, and passed quickly from the position of Rodman to that of Leveler and Engineer in charge of location on various railroad lines in Missouri and Kansas. He also served as Engineer in the Land Department of the Texas Pacific Railroad until 1874, when with a view to a partnership in a prominent law firm, he began reading law. He did not persist in his law studies, however, preferring to accept an opportunity to go to Brazil on bridge construction work. A year later, Mr. Wagoner was employed as Principal Assistant with the Brazilian Carta Itineraria on geodetic work, and continued thereafter, for a

* Memoir prepared by Messrs. C. E. Grunsky, William Hammond Hall, and Otto von Geldern, Members, Am. Soc. C. E.

time, in the private practice of engineering in Brazil. In 1878, he went abroad and traveled quite extensively in Europe.

Returning to Kansas in 1879, Mr. Wagoner found employment in the construction of various municipal water-works. In 1880, he yielded to the lure of California and devoted himself thereafter, principally to mining. In 1881, he was engaged as Mining Engineer for the New Almaden Quicksilver Mine (California) and designed and built, for this mine, a mechanical furnace draft. The next year (1882), he designed and built at Murphy's, Calif., air classifiers which, at that time, were novelties. For a year, he served as Manager of a mine at Alamos, Mexico, but the call of California was strong, and together with the late James D. Schuyler, M. Am. Soc. C. E., he entered the contracting field and built the rock-fill section, No. 5, of the sea-wall at San Francisco, Calif.

From 1885 to 1890, Mr. Wagoner was active in the practice of general mining engineering, with headquarters at San Francisco. In 1890, he accepted the position of Engineer for the Modesto Irrigation District, and it fell to him to submit a competitive design for the La Grange Dam on the Tuolumne River. This dam which raises the water level of the river more than 100 ft., serves both the Modesto and the Turlock Irrigation Districts, and the engineers for these Districts were selected to prepare the competitive plans for the dam. The plans were submitted to the late Col. G. H. Mendell, Corps of Engineers, U. S. A., M. Am. Soc. C. E., who recommended the one prepared by Mr. Wagoner, with some modifications. Due to some misunderstanding with the management of the District, however, Mr. Wagoner resigned, and the construction work following these modified plans was in charge of other engineers.

Among other work done by Mr. Wagoner in the next few years, the building of a concentrator and power plant for the Washington Reduction Works, the making of plans for an irrigation works on the Lower Colorado River, and the design and construction of works for the Channel Bend Mining Company, in Eldorado County, California, may be noted. In 1900, the City Engineer of San Francisco placed Mr. Wagoner in charge of the field studies and surveys of the available sources of water from the Sierra Nevada Mountains. The outcome of the studies made at that time led to the selection of the Tuolumne River (the Hetch-Hetchy Project) as the most available mountain source of water for San Francisco. From 1902 to 1915, Mr. Wagoner was active on reclamation work—levee building and drainage—and irrigation work in California, New Mexico, and Arizona.

It was during this period that he, with Col. W. H. Heuer, U. S. A. (*Retired*), M. Am. Soc. C. E., was called on by the Federated Harbor Improvement Association of San Francisco to report on the commerce and docks of the harbor and to submit a plan for harbor development. For this study, he went abroad again, and returned particularly impressed with the local lack of port facilities for handling freight, as compared with that at the leading ports of Europe. In 1915, on his selection for the purpose by the Department of State, at Washington, D. C., and the Judge Advocate of the Army, came the call from President Menocal of Cuba to take charge at Havana, Cuba, and adjust

the difficulties and controversies which had resulted there in connection with the construction of roads and sewers. Not only did Mr. Wagoner successfully accomplish this mission, but before he returned to California, to recuperate from ills contracted in Cuba (1921), he had the satisfaction of directing the new work that cost between $1 000 000 and $2 000 000.

Mr. Wagoner's attainments in the higher mathematics were particularly noteworthy. He delighted in taking recourse to Calculus in the solution of intricate problems. This appears, for example, in the paper on "Stresses in Curved Masonry Dams", which he contributed, with Mr. Hubert Vischer, to the Technical Society of the Pacific Coast in 1899.

That he had attained a high standing in his profession and commanded the confidence of his fellowmen is sufficiently shown by his selection for the difficult work in Cuba; but as early as 1882 this had already been made evident by his selection by the Governor of California to act as an expert in a controversy relative to the quantity of material placed by the contractors in a section of the San Francisco sea-wall which was being built on a yielding mud foundation.

Mr. Wagoner was a Past-President of the Pacific Association of Consulting Engineers, and of the San Francisco Section of the Society. He was the first President of the Association of Members of National American Engineering Societies in Cuba, and served for many years as a Trustee of the Mechanics Mercantile Library of San Francisco. He was also a Member of the American Institute of Mining and Metallurgical Engineers and of other organizations.

In his passing, the Engineering Profession has lost a valuable member. His friends miss a delightful and loved companion, and the country has lost a good citizen. He is survived by his widow.

Mr. Wagoner was elected a Member of the American Society of Civil Engineers on May 2d, 1906.

FRANK SHERMAN WASHBURN, M. Am. Soc. C. E.*

DIED OCTOBER 9TH, 1922.

Frank Sherman Washburn, the son of Elmer Washburn, a former President of the National Livestock Bank, of Chicago, Ill., was born on December 8th, 1860, at Centralia, Ill.

In September, 1878, he entered Cornell University where be began his engineering studies. From July, 1879, to September, 1880, he was engaged on surveys for the Chicago and Northwestern Railway Company. He then resumed his studies at Cornell, from which he was graduated in June, 1883, having spent the summer months of each year on railroad construction and permanent way work.

In July, 1883, Mr. Washburn entered the service of the Chicago and Northwestern Railway Company, and was engaged on permanent way work until

* Memoir compiled from information furnished by Bruce Barton, Esq., and on file at the Headquarters of the Society.

September, 1884, when he returned to Cornell University for a year of post-graduate work.

From July, 1885, to July, 1887, he was Engineer in Charge of Bridges of the Chicago and Northwestern Railway Company and was engaged in the design, erection, and maintenance of iron and wooden superstructures and masonry. From July to October, 1887, he served as Engineer in Charge of the Wisconsin, Milwaukee, and Northwestern Union, Madison and Peninsula Divisions, of the Chicago and Northwestern Railway Company. He resigned his position with that Company in October, 1887, and gave his attention to the introduction of railway devices of his own invention.

Mr. Washburn re-organized the Belt Line of the Union Stock Yard and Transit Company, and was then sent abroad by the Chicago and Northwestern, the Illinois Central, and several other railroad companies, to study railway economics in Europe. On his return from Europe, in 1889, he took an important part in the development of Purdy's Dam and Reservoir, and other sources of water supply for New York City. Later, he went to Chile as Consulting Engineer in connection with the production and shipment of nitrate of soda. His next important work was as Consulting Engineer in connection with the building of an inter-oceanic canal in Central America, a project which was abandoned when the United States Government undertook the construction of the Panama Canal.

After several years spent in developing coal, iron mining, and water power projects in the South, Mr. Washburn discovered the possibilities of the application of hydro-electric power to the fixation of air nitrogen for the manufacture of fertilizers. The result was the organization of the American Cyanamid Company. Factories were then erected, on the Canadian side, at Niagara Falls, which have since developed an annual production of cyanamid for fertilizer and chemical use valued at more than $10 000 000.

Before the outbreak of the World War, Mr. Washburn realized that if the Chilean supply of nitrate of soda, on which the United States depended for making explosives, was cut off, this country would be in a serious situation in case of war.

As early as 1910, he had considered the possibility of making the United States independent of South American nitrate deposits so that, in case of an enemy blockade, the country could produce its own nitric acid. On the entrance of the United States into the World War, the American Cyanamid Company was requested by the Ordnance Department of the Government to organize the Air Nitrates Corporation to construct and operate the huge $50 000 000 air nitrates plant at Muscle Shoals, Ala. Mr. Washburn was President of the Corporation. The armistice was declared, however, before the Muscle Shoals plant and two others near Cincinnati and Toledo, Ohio, were completed. At the time of his death, which occurred at his home in Rye, N. Y., on October 9th, 1922, Mr. Washburn was Chairman of the Board of Directors of the American Cyanamid Company.

He was married in 1890 to Miss Irene Russell, of Augusta, Ga., and is survived by his widow, a son, and a daughter.

Mr. Washburn was also President of the Goodman Manufacturing Company of Chicago and a Trustee of Cornell and Vanderbilt Universities. He was a member of the American Institute of Mining and Metallurgical Engineers, the American Institute of Electrical Engineers, the Western Society of Engineers, and American Electro-Chemical Society. His clubs included the University, Sigma Chi, Alpha Delta Phi, and Century, of New York City, and the Apawamis Club, of Rye, N. Y.

Mr. Washburn was elected a Member of the American Society of Civil Engineers on November 7th, 1888.

THOMAS DELANO WHISTLER, M. Am. Soc. C. E.*

DIED NOVEMBER 21ST, 1921.

Thomas Delano Whistler, the son of George William and Julia DeKay Winans Whistler, was born in Petrograd, Russia, on January 19th, 1856. He was the grandson of George Washington Whistler who was noted for the construction of the railroad in Russia between Moscow and Petrograd in 1843, and who was succeeded in this work by his son, George William Whistler.

Thomas Delano Whistler received his early education in Europe and in Baltimore, Md. He was graduated as a Civil Engineer from Rensselaer Polytechnic Institute, at Troy, N. Y., in 1881, and in the same year entered the office of the late William E. Worthen, Past-President, Am. Soc. C. E., in New York City, as his Assistant.

In May, 1884, Mr. Whistler was appointed by the New York Aqueduct Commission as Engineer on Surveys, and, in January, 1885, was promoted to be Assistant Engineer on Construction, retaining this position until August, 1886, when he was appointed Resident Engineer on the Colorado Midland Railroad. On the completion of this work, Mr. Whistler returned to New York City, and, in March, 1887, was appointed Division Engineer in charge of the up-town plant of the New York Steam Company where he remained until June, 1887.

Mr. Whistler's last years were spent at Nagold, in the Black Forest, Würtemberg, Germany, where he died on November 21st, 1921. Although he had resided in Germany for a great many years, he had never relinquished his American citizenship.

He was married to Miss Sophie M. Ames, of Cambridge, Mass., who died on August 8th, 1922. He is survived by his daughter, Mrs. Percy D. Haughton, of Charles River, Mass., his brother, Ross Winans Whistler, of Lenox, Mass., and a sister, Mrs. Albert Revillion, of London, England.

He was an ardent and expert fisherman and invented several devices to improve the fishing rod and reel. His exploits in killing sea bass at Newport, R. I., are well remembered.

Mr. Whistler was elected a Junior of the American Society of Civil Engineers in 1884, and a Member on May 2d, 1888.

* Memoir prepared from information furnished by Ross W. Whistler, Esq., and on file at the Headquarters of the Society.

HERMAN DAVID HIRSCH, Assoc. M. Am. Soc. C. E.*

DIED AUGUST 30TH, 1922.

Herman David Hirsch was born in New York City, on October 19th, 1886, and received his preliminary education in the public schools of that City. After his graduation from High School, he won a scholarship to Cornell University, and was graduated with honors from the Department of Civil Engineering in 1909.

Mr. Hirsch entered the employ of the American Bridge Company in July, 1909, at the Pencoyd Plant, as Structural Steel Draftsman, where he remained until October, 1910. He was then transferred to the Brooklyn Plant, and, in November, 1911, he was transferred to the New York Office, where he was engaged, as an Assistant Engineer, in making designs and estimates on highway and railroad bridges and miscellaneous steel structures.

Some of the most important works on which Mr. Hirsch was engaged during his career as an engineer were designs for the emergency dams for the Panama Canal, a transfer bridge for the Long Island Railroad, at Bay Ridge, N. Y., and steel boats for the United States Government at Hog Island, Philadelphia, Pa., during the World War. He also perfected the designs for the military bridges in France.

Mr. Hirsch served in the capacity of Assistant Engineer with the American Bridge Company until September 15th, 1919, when he became associated with the United States Steel Products Company, and was sent to Cape Town, South Africa, as Resident Engineer, which appointment he held until the time of his death. He is survived by his widow and two daughters.

Although his career was cut short by his untimely end, Mr. Hirsch lived to fulfill his promise as a brilliant student, always discharging the duties of his various positions with great satisfaction and being held in high esteem by his associates for his professional ability. He was a man of sterling qualities and integrity of character, conscientious, upright, frank, and sincere, loving his work with singular devotion. His personality was winning and attractive, and all who came in contact with him were pleased to number him among their friends.

Mr. Hirsch was elected an Associate Member of the the American Society of Civil Engineers on June 18th, 1918.

HUGO JULIUS SCHEUERMANN, Assoc. M. Am. Soc. C. E.†

DIED JULY 20TH, 1921.

Hugo Julius Scheuermann was born at Suvalki, Russia, on February 11th, 1873. After completing his elementary education, he attended High School from 1885 to 1889, taking a preparatory course for Engineering Science at

* Memoir prepared by William H. Chorlton, M. Am. Soc. C. E.

† Memoir prepared by William Russell Davis, Assoc. M. Am. Soc. C. E.

the Standard Government School, at Petrograd, from which he was graduated in 1892, with the degree of C. E.

After his graduation, Mr. Scheuermann was appointed Assistant Engineer with the Department of Railroads, Petrograd, with headquarters at Bialystock. He came to the United States early in 1894, and, for two years, was employed with the Southern Pacific Railroad Company on track relocation.

From 1896 to 1904, he was active in the design and construction of structural steel with the American Bridge Company, Hitchens and Company, and Milliken Brothers. In 1904, he was appointed Bridge Designer in the office of the State Engineer and Surveyor of New York State, and held this position until the time of his death, at his home in Albany, N. Y., on July 20th, 1921.

Mr. Scheuermann became a citizen of the United States and married here. He is survived by three children, his wife having died several years ago.

Of a quiet and unassuming personality, he was a conscientious and thorough workman, and was devoted to his family.

Mr. Scheuermann was elected an Associate Member of the American Society of Civil Engineers on December 7th, 1904.